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PAPERS AND DISCUSSIONS

OF GENERAL INTEREST

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JOURNAL OF THE

BOSTON SOCIETY OF CIVIL ENGINEERS

Volume 42 JULY, 1955 Number 3

ETHICS IN ENGINEERING

By Waldo G. Bowman*

(Presented at a meeting of the Boston Society of Civil Engineers, held on November 27, 1954.)

No more knowledgeable or thoughtful audience for a discussion of this subject could be found than is afforded by this venerable society, traditionally noted for its service to engineering ethics and, indeed, for an outstandingly progressive attitude in all things pertaining to the welfare of the civil engineering profession. As a member of that profession, and as one who is grateful for the unearned privilege of basking in the glory of its achievements, I want, more than anything, for it to deserve and live up to its claims and rights of professionalism. It is for that reason that I accepted your kind invitation to discuss the subject of ethics in engineering, which I shall do with the greatest seriousness and sincerity that I can muster.

Actually, I think our concern is more with codes of ethics than with ethics themselves. Each of us knows the difference between right and wrong, including the difference between the right and wrong way to secure an engineering engagement, but it is when an attempt is made to express ethics in words that we run into difficulties. Not only do words sometimes convey different meanings, but the meanings may change with the passage of time.

Because we aspire to be a profession, we engineers must have some special requirements for our codes of ethics, which, incidentally, I would prefer to call professional rules of conduct as one of the ways to avoid our difficulty with words. In any event, while business codes can often be founded on the Golden Rule—furnishing good conduct in return for good conduct—a profession's code must have a much deeper meaning. It cannot confine its concern to the two principals in a transaction, the buyer and the seller, but it must take into ac-

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count the public interest—actually must be focused on the public interest as a prime requirement. Moreover, in addition, a profession's code or rules of conduct must, to be successful, be acceptable to the public as well as to the profession itself.

Thinking then of our rules of conduct as being effective and proper only when they are drawn in the public interest, and when the public thinks they are so drawn, it is interesting to examine the situation with respect to one of the provisions in our codes that has recently been accorded considerable attention. This is the provision in the code of the American Society of Civil Engineers that says it is unprofessional "to participate in competitive bidding on a price basis to secure a professional engagement." And the code of the Boston Society of Civil Engineers is similarly worded: "It is unethical for an engineer knowingly to compete for services to clients on the basis of price."

Properly interpreted, these code provisions state a fine principle. It is one that I personally subscribe to because it reflects my belief that engineering services cannot be bought or sold on price alone, and, indeed, that price is subordinate to the experience and ability of the engineer. Distinctly, it is not in the public interest to make the lowest possible price a measure for engineering services. That, I am sure, is what we interpret the code provision to mean.

But the danger is not in what we think the provision means. It lies in the way we apply it under the pressures of competition. And, most important, the danger lies in the way the public thinks it is applied.

It is this very basic consideration that makes it impossible to regard with complacency a growing tendency for engineers to skate along the edges of violation of this code provision—and in some cases to violate it in fact as well as in spirit. Should this tendency continue, which in effect would be an admission that the code provision could not be lived up to, some means will have to be found to make competitive bidding acceptable, or the wording of the code will have to be changed to cause it to reflect more clearly and convincingly just what is and what is not in the public interest.

In any event, this problem of bidding would seem to be serious enough to warrant some new attention—to determine, at least, why it is a problem and what, if anything should be done about it.

It might seem surprising that a prohibition against competitive bidding for engineering services did not appear in the original code of the ASCE, when it was drawn up in 1914. But it must be recalled that the consulting engineering business of those days was quite different than now. Many of the first consisted only of a principal. A partnership then meant two people, not twenty. Compared to the business available there were relatively few firms. The activities were centered much more on giving advice than in making designs. Under the circumstances, there was only a minimum amount of competition, and bidding in any kind of a formal sense was not even thought of.

As the country grew and engineering expanded with it, consulting firms had to become larger and more numerous. They supplemented their advice with complete design and construction supervision service. They became organized businesses.

And because of their numbers and their size, consulting firms also became more competitive. They had to go out and seek business, rather than waiting for it to seek them. Also, as jobs grew larger, the amount of money paid the consultant became large enough that the owner began to notice it. Price became important to him, and it came more and more into the discussions of the owner and the consultant.

Despite these pressures, however, the consultant clung to his professional principles as best he could; and finally he reinforced his stand in their favor by adding a prohibition against price competition in his societies' codes. This was admirable and proper because the engineer's responsibility to his client and to the public had not changed, even though business practices had been altered.

In this connection it is sometimes argued that our codes of ethics are focused on the consulting engineer, and are, therefore, not a proper document for a society with such a varied membership as the ASCE or even the BSCE. Obviously an organization consisting solely of consulting engineers could live more closely under such codes and administer them more effectively. But it is my belief, nevertheless, that the codes are worthwhile for our kinds of societies because they express the kind of ideals that a civil engineer, no matter the character of his job, needs to remind him that his calling is one of service to the public and mankind. The spirit of these codes is the right one for every engineer to live by.

But the public also needs to be reminded and convinced that the engineer has these ideals, and tries to live up to them. It is for this reason that deviations from the code provision against competitive price bidding are so important. They tend to raise doubts in the public mind that engineers really cherish these ideals. And under present circumstances and the wording of the codes there is no way for the public to know when one form of bidding is proper and another is improper. Actually the public is right in assuming that all shades of bidding are considered wrong because that is what the code says.

It seemed easy to decide that when South Carolina, a year or so ago, called for bids for the design of a bridge, and when thirteen firms responded, that this was a case of bidding on a price basis. Actually it cannot be said with finality that it was not in the public interest, yet such an open invitation for bids at least opens the opportunity for a too-low-price award, which could result either in an extravagant design or an unsafe one.

In another case, however, the Ohio Highway Department, asked four selected firms to submit proposals, naming price, in sealed envelopes for a turnpike study job. Three firms responded. Since the firms were presumed to be of equal competence, the Ohio highway director had little except price to base his choice on. He assured these firms that he was not taking bids, but just getting proposals as a basis for further negotiation. Yet it is not he who determines whether this would be classified as bidding on a price basis, but a committee of the ASCE.

As a matter of fact, the South Carolina engineer also said that he would not base his selection on price alone but would take ability and experience into account.

This matter of deciding who is in violation of the code and who is not is one of the most difficult aspects of this bidding problem. It raises the question of whether these society committees should be expected to shoulder the huge responsibility of deciding who is guilty and then assigning the punishment. With no more of a guide than is given in the code I submit that they are being asked to carry too great a load. It is one thing to set punishment for a misdeed that is recognized as such by the public, when, for example, there is dishonesty, negligence and the like involved. But bidding per se is not recognized as an immoral act. Some forms of it may be, but the public is not well enough informed to lend any support to the committee's judgment.

It seems clear that some way must be found for the profession to live with and live up to this code provision on bidding. There are those who maintain that it is being honored in 99% of the cases, and that the 1% of violation is therefore negligible. If only the plain and extreme violations are counted that may be the case, but when one takes into account the charges and recriminations that engineers bring against one another's business practices, one cannot be so sure. The sad part is that most of the practices that are complained about are neither wrong nor harmful. They are just, according to someone's interpretation, in violation of the wording of the code.

In the desire to prohibit the kind of price competition that would depreciate engineering services and harm the public, the prohibition has been so worded as to make all shades of price competition—even competition itself—suspect.

Let us by all means keep and cherish our engineering ethics. But let us also keep the rules of conduct that express them bright and shining. Sometimes this may require some polish on a rule that has become tarnished. It is my belief that the rule relating to competitive bidding is in need of some polish—in order that it may express truly and plainly what the profession means to live up to. Once we delimit the prohibition to the kind of price competition that is demonstrably harmful to the public, not only will the profession have a better chance to live up to it, but the vitally necessary public support can be generated to enforce it.

DISCUSSION

BY THOMAS R. CAMP,* *Member*

YOUR President has asked me to discuss the history of the development of the Society's Code of Ethics and more particularly the more recent developments relating to bidding for engineering services. The Code of Ethics of the Boston Society of Civil Engineers was adopted on December 18, 1912. No revisions have ever been made in the 1912 printed Code of Ethics although steps have recently been taken by the Society which require revision of the code.

The Canons of Ethics for engineers adopted April 8, 1947 by the Engineers' Council for Professional Development were referred to the Boston Society of Civil Engineers with the request that consideration be given by the Society to the adoption of the Canons of Ethics for the Society. A committee which was appointed by the

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Board of Government to consider the matter reported in favor of keeping the Society's 1912 Code of Ethics and of notifying the E.C.P.D. of the approval by the Board of Government of the principles set forth in the Canons of Ethics. The Committee recommended that both the Code of Ethics of the Boston Society and the Canons of Ethics be published in the JOURNAL.

The matter of bidding on a price basis for engineering engagements was brought to the attention of the Society in the presidential address of George A. Sampson in March 1947. A committee on competitive bidding was appointed to look into the matter by the Board of Government. After two years' deliberation, the committee drafted a brochure "Recommended Procedure for Securing Professional Engineering Services by Public Authorities" which was endorsed by the Society on November 9, 1949. This brochure cites the evils of competitive bidding and describes a procedure by which an engineer may be selected by negotiation. Copies of this brochure were mailed to municipal and state authorities throughout Massachusetts.

In 1950 during my tenure as president a new committee on professional ethics was appointed and directed to re-examine the whole matter of revision of the 1912 Code of Ethics in the light of the E.C.P.D. Canons of Ethics, the recent revisions of the A.S.C.E. Code of Ethics and the recent action of the B.S.C.E. on bidding for professional services. The A.S.C.E. had on November 1, 1949, revised its Code of Ethics to include a provision against competitive bidding on a price basis to secure a professional engagement. At the annual meeting of the B.S.C.E. in 1951, the new committee on professional ethics reported in favor of retaining the Society's 1912 Code without change. At the same meeting the Committee which had been laboring for some time on revisions of the constitution and by-laws reported in favor of amending the Code of Ethics to include a provision against bidding.

During 1951 a new committee was appointed and directed to consider the conflicting recommendations which were made at the preceding annual meeting and advise the Board of Government as to what action should be taken. This committee recommended that the following resolution should be adopted by the Society:

> "Resolved: That it is undesirable for an engineer knowingly to compete for services to clients on the basis of price."

Action was taken on this resolution at the meeting of the Society in September 1952. The resolution was amended by substituting for the word "undesirable" the word "unethical". The resolution was adopted by vote from the floor. The amended resolution was then sent out to letter ballot. The results of the letter ballot were canvassed in October 1952. Of 290 ballots received, 266 or 92% were in favor, 22 were against, and 2 were blanks. I am strongly of the opinion that this vote constitutes a directive to the Board of Government to amend the Code of Ethics to include a provision against bidding for engineering engagements. Nothing has been done to date to amend the code to conform with this vote.

If engineering is to be considered by the public as one of the learned professions, engineers must themselves act as members of a learned profession should act with the interest of the public paramount. I presented my views on engineering ethics at some length in my presidential address, which was published in the April 1951 JOURNAL of the Society. To repeat some of that address, "It is my conviction that it is not in the public interest to engage engineering services on the basis of competitive price bids because, (1) the professional nature of the services makes it impossible to specify the type, extent and quality of services upon which bids are to be received, (2) no two engineers are equally qualified for a particular engagement, (3) the practice automatically deprives the public of the services of some of the best qualified engineers who do not have to secure engagements in this manner, and (4) if price is made a consideration it is almost certain to become the most important consideration to the awarding agency". If it is not in the public interest to engage engineering services on the basis of competitive price bids, it is unethical for engineers to compete on this basis. In my opinion engineering is at a crossroads and the stand we take on competitive bidding is going to determine to a considerable extent whether we attain true professional status in the public eye.

Mr. Bowman has pointed out a very pertinent question in the matter of bidding for engineering engagements. Some engineers profess to believe that a formula can be developed whereby several proposals can be submitted containing prices without violation of ethical principles. This is complete nonsense. If more than one proposal containing price is submitted to a client before he selects his engineer, it constitutes bidding on a price basis. If we value our professional status, we must face up to this issue courageously.

DISCUSSION

BY THOMAS A. BERRIGAN,* *Member*

TO COMPETITIVELY bid or not to bid for engineering services—that is the question which was brought into sharp focus by an advertisement which appeared through inadvertence in the *Engineering News-Record,* and was commented upon editorially in that same magazine on November 19, 1953.

The editorial referred to invited attention to a procedure which is considered to be in accord with code provisions of the American Society of Civil Engineers and which provides that prospective clients interview a number of invited engineers personally, receive their proposals and weigh their qualifications. The editorial takes the stand that this procedure should be broadened to permit engineers to compete on a price basis, providing that price alone is not the determining factor in the final selection. The editorial also contends that unless the code is broadened to permit engineers to competitively bid, professionals of high moral character will circumvent (Will they—and if they do—are they?) the provisions of the code to stay in business. The stand taken in the editorial is further justified on the ground that many state laws require competitive bidding, and that competitive bidding will minimize the practice of award of engineering contracts to incompetent and unscrupulous engineers providing the awarding authority carefully regards the qualifications of the bidders.

Keeping in mind the principles of right conduct in the accomplishment of engineering projects, I feel strongly that the client is the principal party in interest, and that quality of service is the thing to be desired most. In view of this, I subscribe to the proposition that selection is the essence of the professional contract, and find that bidding for services such as obtains in the acquisition of commodities is not conducive to the best end result. Equally important is the question of "who makes the selection". Obviously, if the selection is made on the basis of a recommendation of an engineer charged with a trust in the public service, the client's interest will be better served than if the selection is made by individuals who, by training, experi-

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ence or disposition, have no capacity to make a proper selection, notwithstanding the fact that they have the legal capacity to consummate the contract.

The price to be bid for engineering services is not, in my opinion, too important because differences in price do not add materially to the cost of the project. I am inclined to regard these differences in price as infinitesimals of a higher order—with all due apologies to my professors in mathematics who repeatedly advised me in my early days that such terms are applicable only to the product of infinitely small increments.

Fortunately in Massachusetts there is no legal requirement to invite bids for professional services and it is my urgent recommendation that the professional societies endeavor to change the law in states where the requirement exists, rather than subscribe to the proposition that the code be revised for expediency.

Unquestionably, some hardship will be imposed because of loss of business among those professionals who refrain from competitive bidding. However, I think the profession of engineering will ultimately be strengthened and thereby hold its proper place with medicine and law.

Because selection is so important, I am suggesting that consideration of price for engineering services be postponed until after the engineer has been selected. Under this arrangement a price which is fair and equitable for the services to be rendered can be negotiated, and the client and the professional engineer will be equitably treated.

Competitive bidding for professional services is an attempt on the part of certain agencies to acquire personal services at the lowest price notwithstanding allegations to the effect that public officials are relieved of political pressure, and that incompetent and unscrupulous engineers can be disqualified readily. Professional services of an engineer on a sizeable project are just as unique and special as they are in medicine, law or the theatre. Contemplate, if you will, the performance of a Drew or Barrymore in his own special field; the services of a specialist in brain or lung surgery; and the capacities of different lawyers in tax matters or specialists drafting bond issues covering the financing of hundred million dollar toll road programs. Relief of a public official from political pressure which is not in the public interest is an admission of moral deficiency. If, in fact, such a moral deficiency is apparent, there is no assurance that incompetent engineers will be excluded from selection. Further, engineering and architectural contracts touch upon a service which is so personal that language in the instrument permitting successors or assigns to perform the contract is improper. In this connection may I invite your attention to the fact that many states deny corporations the right to practice engineering and that law firms are never incorporated because of the personal nature of the service.

POLLUTION ABATEMENT PROGRAM AT NEWPORT, RHODE ISLAND

By Edwin B. Cobb* and Dean F. Coburn,** *Members*

(Presented at a meeting of the Sanitary Section, B.S.C.E., held on December 1, 1954.)

The public sewerage system of Newport, Rhode Island, was begun over 80 years ago. Increasing population in the city, together with the ever increasing naval activity in the area, outmoded the practice of disposing of the sewage in Narragansett Bay. After 7 years of study and development, most of the sewage from the City and the principal naval facilities will be treated in a modern sewage treatment plant. This paper will describe the more important phases of the pollution abatement program.

The city of Newport is located on a peninsula forming the southwest portion of Aquidneck Island at the mouth of Narragansett Bay. Aquidneck Island is about 16 miles long and about 5 miles wide.

The topography of the City is generally hilly, with elevations along the central axis of the peninsula which reach 100 to 159 ft. above sea level. The shore line is irregular, characterized by rocky headlands and coves with sandy beaches.

Newport is rather sharply divided into distinct types of development. The southwest half of the peninsula is entirely occupied by palatial summer estates, exclusive golf clubs, and bathing beaches. In general, the central part of the peninsula west of the ridge line is densely built up with residences and commercial establishments, while east of the ridge line the area is rather completely developed with new-style residences. In the northern section of the City are the naval installations, which follow along the west shore; east of these are several hundred acres of undeveloped land. The City incinerator and dump occupy a portion of this vacant land, and in recent years small commercial developments have been pushing into the area.

Eastons Pond, a principal source of potable water for Newport and the adjacent town of Middletown, occupies the southeast corner of the City. Adjacent to the pond is Newport Beach, a large expanse of smooth sandy beach providing excellent surf bathing.

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Adjacent to Newport to the north and east is the town of Middletown, which occupies the central portion of Aquidneck Island. The town of Portsmouth occupies the northerly portion of the island. Portsmouth and Middletown are suburban-agricultural communities and contain many high-class farm estates. These towns may be considered as suburbs of Newport. There is a summer resort and amusement center at Island Park in Portsmouth and a growing summer-cottage colony at Eastons Point in Middletown.

Along the west shore of the island, extending from Newport to Portsmouth, is a continuous development of U. S. Navy installations. Coaster Harbor Island, Goat Island, and Rose Island lying off this shore are also entirely occupied by the Navy.

Newport had its beginning in early colonial times and has grown steadily in spite of the lack of any important industrial, agricultural, or shipping activity. The 1950 United States Census reported the City to have a population of 37,564 exclusive of military personnel. During World War II the military population was estimated as in excess of 40,000, but the present military population is much less although it varies considerably.

The trade, various services, and other activities in connection with the summer colony and Navy provide the principal sources of livelihood in the City. An electrical manufacturing concern operates two medium-sized plants in Newport, and there is also a fishing industry. Each year, many tourists visit Newport, attracted by its famous estates, the shore drive, beaches, and historical landmarks.

The climate at Newport in the summer is tempered by the sea breeze, and this together with its comparative isolation from industrial activities has favored its growth as a summer resort. There is, however, no extensive summer cottage colony within the City limits; only a little land is now available for such development, owing to restrictive zoning regulations and existing occupancy.

The naval shore installations on Aquidneck Island in the vicinity of Newport include a variety of activities, the majority of which are concerned with training, the servicing of the destroyer fleet, and the manufacture and servicing of torpedoes. These installations are continuous along the shore, beginning with the Naval Hospital at Cypress Street in Newport and extending to the Naval Fuel Plant at Coggeshall Point (Melville) in Portsmouth, a distance of nearly 7 miles.

The Navy has a private highway which extends the entire length of its reservation.

The Navy also occupies the principal wharves at Newport, and has a ferry landing with adjacent parking in the central business district. Narragansett Bay provides excellent anchorage facilities for naval vessels and since World War II these facilities have received much use.

A general plan for a main sewerage system for Newport was prepared in 1880 by the City Engineer, J. P. Cotton, and approved by J. P. Kirkwood, Consulting Engineer. Mr. Kirkwood was one of the best known sewerage engineers of his time. This general plan was slightly modified in 1885 by City Engineer Henry A. Bentley. The system as constructed utilized several old sewers which had been built before 1875 and which apparently were intended originally as storm drains. Among these were the Broadway sewer which formerly discharged to the harbor at Long Wharf, and the Middleton Avenue sewer which formerly discharged to the ocean at Forty Steps.

The Middleton Avenue sewer was intercepted at Ochre Point Avenue by a trunk sewer which followed Ochre Point Avenue, Webster Street, Lawrence Avenue, and Ruggles Street to a point west of Coggeshall Avenue, thence ran across lots and Morton Park to Morton Avenue, thence by Morton Avenue to Thames Street where it discharged to a trunk sewer which followed Thames Street.

The trunk sewer in Thames Street intercepted the Broadway sewer at Washington Square and thence continued across the railroad yards to Briggs Wharf where it discharged to an outfall sewer. The general arrangement of the main sewers of the city is shown in Fig. 1

The outfall sewer was constructed in 1885, and when originally completed terminated at the north end of Goat Island on what was then an underwater projection. Subsequently the land area of the island was extended by the Navy, and the pipeline covered. The outlet end of the pipe was left flush with the face of a granite sea wall, and its invert about 10 ft. below mean low water.

The outfall sewer was constructed of 30-in. cast-iron pipe with flanged ends and was placed directly on the bottom of the harbor with little or no cover. To enable the pipe to meet the irregularities of the bottom, specially beveled wooden spacers were placed between

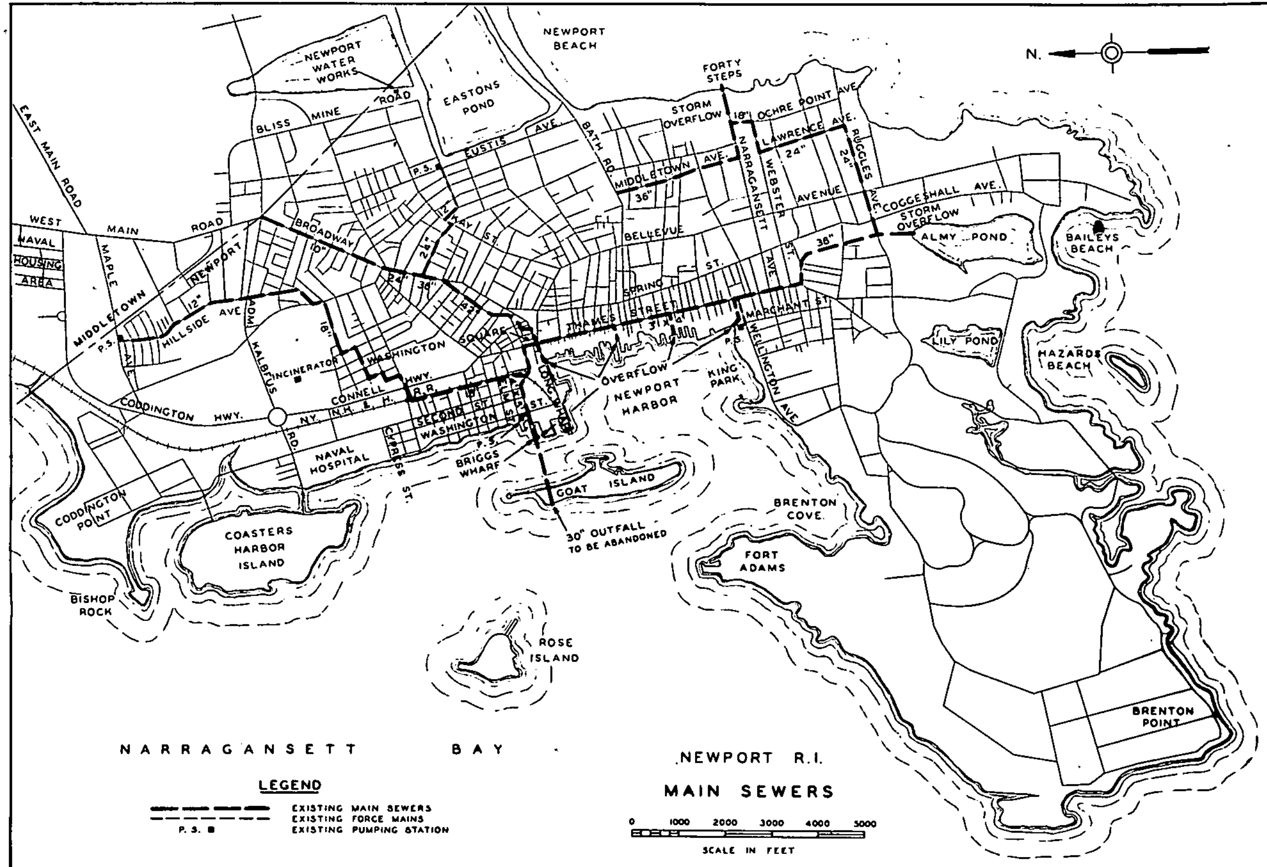

NEWPORT R.I.
MAIN SEWERS
LEGEND
EXISTING MAIN SEWERS
EXISTING FORCE MAINS
P. S. ■ EXISTING PUMPING STATION
0 1000 2000 3000 4000 5000
SCALE IN FEET
N.
NARRAGANSETT BAY
NEWPORT BEACH
FORTY STEPS
STORM OVERFLOW
NEWPORT WATER WORKS
EASTONS POND
ALMY POND
LILY POND
BAILEYS BEACH
HAZARDS BEACH
NEWPORT HARBOR
KING PARK
GOAT ISLAND
30" OUTFALL TO BE ABANDONED
BRENTON COVE
FORT ADAMS
BRENTON POINT
ROSE ISLAND
COASTERS HARBOR ISLAND
CODDINGTON POINT
BISHOP ROCK
NAVAL HOSPITAL
BRIGGS WHARF
INCINERATOR
NAVAL HOUSING AREA
EAST MAIN ROAD
WEST MAIN ROAD
CODDINGTON HWY.
THAMES STREET
SPRING ST.
BELLEVUE AVENUE
BROADWAY
OCHRE POINT AVE.
COGGESHALL AVE.
LAWRENCE AVE.
WELLINGTON AVE.

FIG. 1.

flanges as required. The sewer, about 1,400 ft. in length, crossed a busy channel.

It is reported that the outfall sewer has been broken about ten times since it was laid. The last break occurred before August 1943, and is located near the east shore of Goat Island. This leak has not been repaired.

In 1880 very little knowledge of rates of rainfall or of stormwater runoff existed. Designs were based largely upon the experience in England, where rates of rainfall are considerably less than those experienced in this country; but even there, little definite information was available. Further, there was no information at the time as to the relation between rate of runoff and precipitation. The engineers apparently assumed that, at times of heavy rainfall, water would either accumulate on the surface or soak into the ground to such extent that the amount reaching the sewers would be comparatively slight.

Old reports indicate that the sewerage system laid out in 1880-1885 was designed primarily to carry domestic sewage. It was stated that an allowance was made to take stormwater in an amount equivalent to a depth of ¼ in. over the drainage area in 24 hr. This was a common allowance in the design of sewers at that time, and corresponds to a runoff over the area drained at an approximate average rate of 1/100 in. in depth in 1 hr. Actually, rainfalls of nearly 2.5 in. have fallen in 1 hr. at Providence, and even higher intensities have been recorded over shorter periods.

To provide outlets for the excessive quantities of stormwater reaching the sewers, numerous overflows were constructed in the trunk sewers. In general these overflows function only when storm flow augments the normal sewage flow beyond the capacity of the sewer.

Three overflows were provided in the Thames Street trunk sewer, and other overflows have been constructed on the sewers to permit the discharge of storm flow to the ocean, harbor, or convenient watercourse.

A major overflow of the sewerage system was provided at Washington Square. Side overflow spillways were provided in two of the largest sewers and discharged to a large conduit which led a short distance to shallow water at the inner end of Long Wharf.

Another important overflow was provided at Briggs Wharf at the inlet to the outfall sewer. Stop logs were maintained in this overflow to prevent frequent discharge. When high rates of flow taxed the capacity of the main sewers, the stop logs were removed from the overflow.

The existing sewerage system has shown itself adequate to handle the sanitary sewage of the City under dry-weather conditions. Some difficulty was encountered at the Washington Square overflow under high-tide conditions. At high tide there was little head available to induce flow through the outfall sewer; consequently sewage frequently overflowed at Washington Square even under dry-weather conditions.

Notwithstanding the fact that portions of the system were not designed for and are incapable of carrying stormwater from the tributary areas, attempts have been made to utilize these sewers for stormwater drainage. Many catchbasins and stormwater inlets have been constructed and connected to the sewers, and many roof and cellar drains have also been connected. Flooding has been the natural result.

As a result of complaints caused by overflowing sewers, the firm of Metcalf & Eddy, Consulting Engineers, was retained to review the problem. In 1928, the engineers compiled a comprehensive plan of the then existing sewerage system, and submitted recommendations for improvements and additions to the system. An extensive stormwater drainage system and separate sewers were included in the recommendations. The report also recommended the interception of a number of local sewers which discharged directly to the harbor and the diversion of their flows to the main outfall at Goat Island.

In spite of the Depression, World War II, and public and political apathy, a number of the engineer's recommendations were carried out. Nevertheless some important projects were ignored, while the continued growth of the City (spurred by war activity) intensified some of the the objectionable conditions.

Because of rising public objection to the increasing sewage pollution of the harbor and beaches, and because of Rhode Island's vigorous anti-pollution program, Metcalf & Eddy in 1948 was again retained to review the sewerage and drainage problem of the City.

The tremendous expansion of the Navy at Newport during World War II had also projected the Navy into the pollution problem. All the naval shore installations were equipped with water carriage sewage systems and most of these systems discharged untreated sewage directly to the harbor. As a result of public pressure for action, the

Navy also retained Metcalf & Eddy to review the Navy's own problem of pollution abatement.

By 1948 the majority of the City sewers carrying domestic sewage discharged to the main system tributary to the outfall sewer at Briggs Wharf. However, a few local sewers near the water front still discharged directly to Newport Harbor, and harbor-front buildings which could not be served by existing City sewers were connected to private sewers, which also discharged to the harbor. Sewers in two streets near the west end of Newport Beach also discharged to the ocean. Recommendations to connect these sewers to the main sewerage system had been incorporated in the 1928 Metcalf & Eddy report.

In the naval installations in some locations, the entire activity was served by a single collecting system. In other locations individual buildings had separate outfall sewers which discharged to the bay. Some of the collecting systems received both stormwater and domestic sewage. Other systems received domestic sewer only, with a stormwater drainage system being provided in some cases. In several instances buildings discharged sewage into what were believed to be stormwater drainage systems.

During World War II, sewers were constructed in the section of Middletown bordering West Main Road from the Newport city line to the Naval Housing Area (Anchorage). Middletown was given a revokable permit by the Navy to discharge the sewage from this area into the Navy sewerage system at the Naval Housing Area. It was reported that stormwater also discharged into the Middletown sewerage system.

As the first step in their studies the engineers investigated the waters in the vicinity of Newport Harbor, to determine the extent and magnitude of objectionable conditions. Visual inspections at various points around Newport Harbor were made between February and July, 1949. Locations were selected to show average conditions rather than the worst.

An extensive sewage field was plainly visible as a rule at the main outfall on the west shore of Goat Island. Tidal currents at times swept this field around either the north or south ends of the island into the inner harbor. The average direction and velocity of tidal currents at various stages of the tide are shown in Fig. 2.

The leak in the outfall sewer definitely contributed to the pol-

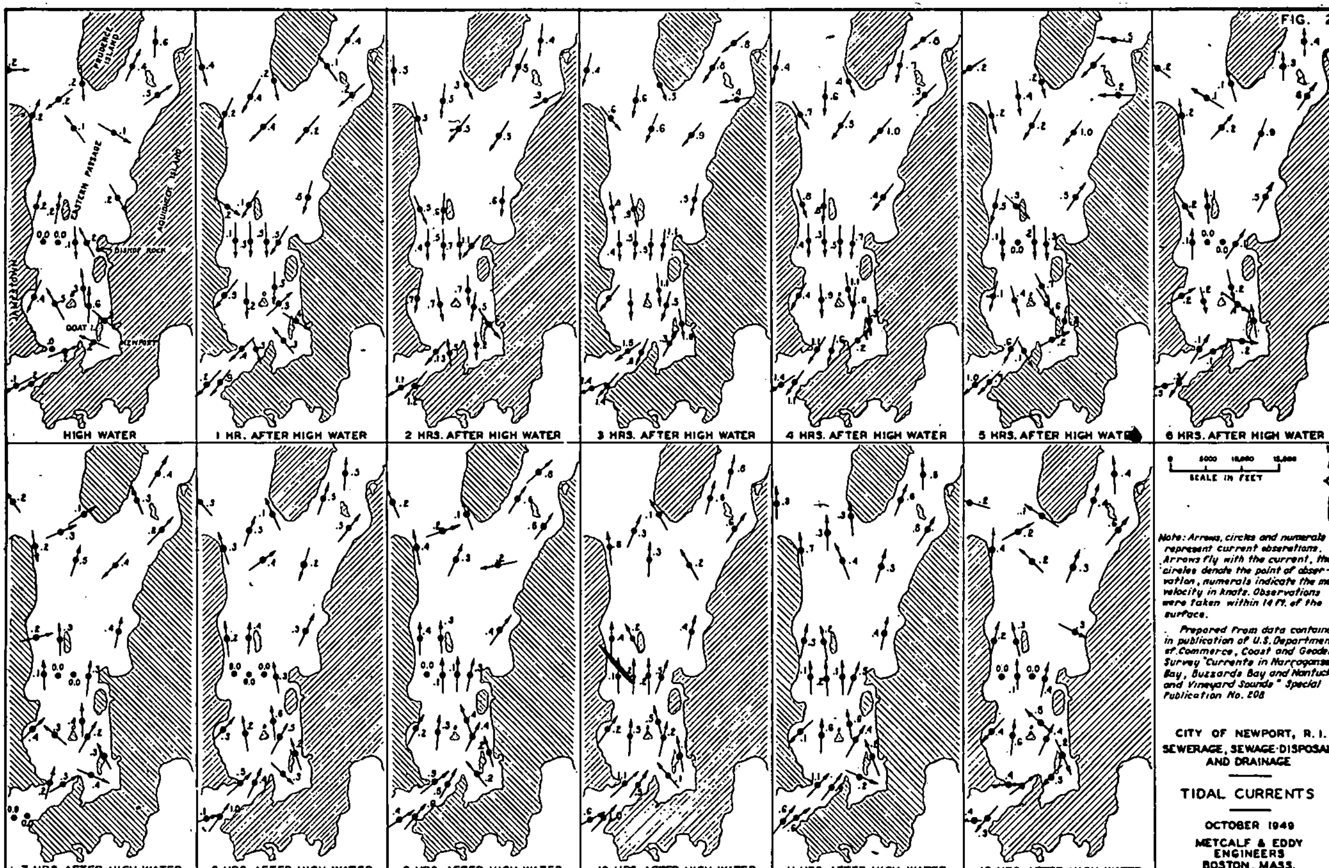


FIG. 2
PRUDENCE ISLAND
EASTERN PASSAGE
AQUIDNECK ISLAND
BISHOP ROCK
JAMESTOWN
GOAT I.
NEWPORT
HIGH WATER
1 HR. AFTER HIGH WATER
2 HRS. AFTER HIGH WATER
3 HRS. AFTER HIGH WATER
4 HRS. AFTER HIGH WATER
5 HRS. AFTER HIGH WATER
6 HRS. AFTER HIGH WATER
7 HRS. AFTER HIGH WATER
8 HRS. AFTER HIGH WATER
9 HRS. AFTER HIGH WATER
10 HRS. AFTER HIGH WATER
11 HRS. AFTER HIGH WATER
12 HRS. AFTER HIGH WATER
SCALE IN FEET
N.
Note: Arrows, circles and numerals represent current observations. Arrows fly with the current, the circles denote the point of observation, numerals indicate the mean velocity in knots. Observations were taken within 14 ft. of the surface.
Prepared from data contained in publication of U.S. Department of Commerce, Coast and Geodetic Survey "Currents in Narragansett Bay, Buzzards Bay and Nantucket and Vineyard Sounds" Special Publication No. 208
CITY OF NEWPORT, R. I.
SEWERAGE, SEWAGE DISPOSAL AND DRAINAGE
TIDAL CURRENTS
OCTOBER 1949
METCALF & EDDY
ENGINEERS
BOSTON, MASS.

FIG. 2.

lution of the inner harbor. Oil sleeks, sewage solids, garbage, and other debris typical of sewage pollution were frequently observed floating along the harbor front, especially in the vicinity of the various local sewer outfalls and at some of the sewer overflows.

Particularly objectionable conditions were often observed at the outlet at the head of Long Wharf which served the Washington Square overflow. This outlet discharged to partially confined water at the inner end of Long Wharf. With southerly or westerly winds, floating sewage solids discharged by the overflow tended to remain in the vicinity, causing an unsightly condition and giving off objectionable odors. On Long Wharf and in its vicinity were a number of commercial establishments and a yacht club. The area was also a convenient mooring place for fishing boats and other small craft. The frequent objectionable conditions of the water near the overflow outlet caused numerous complaints.

Since there were no exposed tidal flats or extensive areas of shallow water within a mile of the outfall at Goat Island, the deposition of sludge from the city's sewage appears not to have caused objectionable conditions. Rather, the objections stemmed from floating solids and from sewage solids stranded along the shore.

During the period from July 11, 1947 to August 7, 1947, inclusive, the Sanitary Division of the Rhode Island State Department of Health made a sanitary survey of Newport Harbor and vicinity.

The amounts of dissolved oxygen present near the surface of the Harbor and also at the bottom were measured at ten representative locations on 6 days. None of the results indicated a condition of serious oxygen depletion.

Samples were also collected from 24 stations on the 6 days of the survey, and determinations were made of the number of Coliform group bacteria present in each sample. As a means of evaluating the results of the bacterial survey, the samples were compared with the classification of waters in shellfish areas established by the U. S. Public Health Service. This classification is arranged in terms of the number of Coliform group bacteria present, and is divided as follows:

Class A—Approved area (0-70 Coli/100 ml.)
Class B—Moderately polluted, restricted area
(71-699 Coli/100 ml.)
Class C—Grossly polluted, closed area
(700 or more Coli/100 ml.)

Only one location was found to be in the approved range and six were in the grossly polluted range. The others were classified as restricted area.

In August, 1948 the United States Navy conducted a bacteriological study of the waters in and about the Naval Base from Coddington Cove to Long Wharf. Samples were collected from seven different stations twice daily until a total of 49 samples had been gathered. All samples were given the complete test for Coli organisms, and these bacteria were found in all samples. From seven different samples, the pathogen Shigella Dysentariae (a type of bacteria causing dysentery) was isolated.

In March and April, 1949 Metcalf & Eddy conducted a pollution survey from King Park in Newport to Melville in Portsmouth along the shore of Aquidneck Island, including the waters adjacent to Gould, Rose, and Goat Islands. Two trips over the area were made each week, with a total of eight for the survey. The physical appearance of the water was noted and a bacterial sample collected at each of 21 stations on each trip. The concentration of Coli-aerogenes was determined in each bacterial sample. The location of the sampling points and a summary of the bacterial analyses are given in Table 1.

These sampling points were selected to give representative indications of the general condition of the water. Samples taken in close proximity to outlets discharging untreated sewage will show high pollution, but such information gives little indication of the extent of the polluted area.

Most of the observations indicated no floating or suspended matter except in the vicinity of the Coddington Cove and the City outfalls. Some floating matter was reported in the vicinity of ships at anchor in the Bay.

The analyses of the bacterial samples from Stations 13 to 21 are of special significance to the City since they are in the vicinity of Newport Harbor. The maximum Coli concentrations at all of these stations are in the ranges of either moderate or gross pollution, as established by the U. S. Public Health Service. The median of the series at each of these stations falls within the moderately polluted range, with exception of the samples in Brenton Cove, those close astern the repair ship "Vulcan" anchored off Goat Island in strong tidal currents, and those near Rose Island. The samples taken near the City outfall consistently showed gross pollution.

Samples collected over the City outfall sewer, between Briggs Wharf and Goat Island, showed high bacterial concentrations generally well beyond the minimum limit for gross pollution. These results indicate the effect of the leak in the outfall sewer on the waters of the inner harbor.

TABLE 1

Bacterial Pollution—Narragansett Bay (March 8 to April 4, 1949)

Sta.	Location	Most probable number of Coli bacteria, per 100 ml. Maximum	Minimum	Median
1	100 yd. south of Gould Island	21	0	3.6
2	100 ft. north of North Pier Gould Island	8.8	0	3.3
3	Buoy "N14A" north of Melville	38	5.0	8.8
4	100 ft. of entrance to basin at Fuel Plant Melville	240	8.8	38
5	Black buoy "Cl" southeast of Dyer Island	38	0	8.6
6	100 ft. off dock at Midway	240	2.0	12.1
7	Buoy "N8" off Topedo Station Annex	38	5.0	26.5
8	Red buoy at north anchorage, Coddington Cove	620	8.8	139
9	300 ft. west of Coddington Cove outfall	700,000	38	43,000
10	Buoy "N2" northwest of Coddington Point	620	8.8	38
11	Bell buoy "8A" off Bishop Rock	620	2.2	29.5
12	100 ft. offshore on north line of War College	230	0	38
13	Mooring buoy "MT12" opposite Naval Hospital	620	130	230
14	Over City outfall midway between Briggs Wharf and Goat Island	700,000	230	4,700
15	100 yd. north of spar buoy opposite bathhouse at King Park	2,400	230	430
16	Center of Brenton Cove opposite South Dock	240	5.0	96
17	Black buoy "1" southwest at Goat Island	24,000	5.0	620
18	100 ft. astern repair ship "Vulcan"	620	15	38
19	500 ft. astern repair ship "Vulcan"	24,000	15	235
20	Black buoy "3" southeast of Rose Island	240	8.8	29.5
21	100 ft. west of outlet to City outfall at Goat Island	2,400,000	38	470,000

A concentration of 1,000 Coli-aerogenes in 100 ml. of water is considered by many health authorities as the safe limit for bathing waters. The survey indicated that on at least two occasions the concentration in the waters off King Park beach greatly exceeded this limit. The median of all samples in this location was 430.

From the above surveys, it appeared that the waters from Coddington Cove to Fort Adams Light were bacterially polluted to some degree. While not all the bacterial pollution was from the City outfall, it was a contributing agent and discharged by far the greatest number of bacteria into the waters of the harbor.

In December, 1946 the Rhode Island State Department of Health submitted a report to the Governor, classifying the waters of the State in accordance with their sanitary condition. The classifications with their limits as defined for salt waters were as follows:

Class A—Waters suitable for cultivation of market shellfish.
Class B—Suitable for bathing.
Class C—Suitable for recreational boating, fishing, and the culture of seed oysters.
Class D—Suitable for use primarily for commercial navigation or transportation of wastes without nuisance.
Class E—Grossly polluted, causing nuisance.

In the above report, the waters bordering Aquidneck Island from Arnold Point in Portsmouth to Fort Adams Light, and extending into Narragansett Bay to surround Goat Island and Gould Island, were indicated as Class C. The remaining waters surrounding Aquidneck Island were indicated as Class A.

A large area of the waters of Narragansett Bay above Aquidneck Island also were classified as Class A in the report. Since this area was interposed between the polluted waters adjacent to Newport and the polluted waters of the Providence River and of Mount Hope Bay, the inference was that any pollution occurring at Newport was, in the opinion of the State Health authorities, of local origin and not attributable to conditions upstate.

On the strength of the information at hand, the State Department of Health had prohibited the gathering of shellfish for human consumption from the area between Fort Adams and Coddington Cove. The Department had also warned the City that bathing at Kings Park beach was hazardous. The Navy had closed its beach at Coddington Cove.

After considering all the above data, the engineers in their report to the City summarized the effects of sewage pollution in Newport Harbor as follows:

"Sewage discharged from the main City outfall sewer both at its outlet and through its leak, and that discharged at the Washington Square overflow, and at the various individual small outlets, contributed to the unclean appearance of the water in parts of Newport Harbor. The dirty appearance of the water makes the waterfront unattractive for residential or commercial occupancy, discourages recreational boating in the harbor, and spoils the beauty of waterfront parks. Dirty water fouls the appearance of vessels and especially in the case of yachts increases the difficulty of maintenance. Floating and suspended solids in water increase operating difficulties where harbor water is used for engine and condenser cooling water, both afloat and ashore.

"The eating of shellfish from sewage-polluted water, is extremely dangerous. We are informed that a good quahog area exists in Brenton Cove. The gathering of these shellfish is prohibited by the State Department of Health because of the pollution.

"Bathing in sewage-polluted water is hazardous. During the summer months a considerable number of persons bathe at Kings Park beach and a small number bathe in the waters along Washington Street. A majority of the bathers are children who are ignorant of the hazards involved by bathing in polluted water. The commonly accepted limit for bacteria of the Coliform group in bathing waters is 1,000 per 100 ml. The various surveys have indicated high bacterial pollution in the vicinity of Briggs Wharf and that at times the bacterial concentration at Kings Park is well over the safe limit. Bathing in Newport Harbor is hazardous from the health viewpoint.

"High bacterial pollution of harbor water presents a hazard to the health of personnel on board vessels anchored in its waters. On many vessels equipment is provided for the distillation of sea water into potable fresh water. Improper operation of such equipment, to the extent that foaming or short-circuiting occurs, may result in the production of a distillate of such high salt content as to be undrinkable. There is the possibility that some pathogens may also survive and be carried into the distillate by the foam. Further pollutional hazards occur on shipboard through the various uses of salt water about the vessel. Salt water is often used for washing clothes, bathing, removing peelings from potato peelers, and swabbing decks. There appear many ways on shipboard in which polluted salt water may be inadvertently introduced in the mouth.

"To a majority of health authorities and to an increasing proportion of the general public, the discharge of untreated sewage into tidal waters is regarded as an unclean practice. Newport is being deprived of some of the usefulness of its harbor and a number of its people and guests are exposed to hazards of health through the continued discharge of untreated sewage into the harbor from City sewers."

Pollution by Navy sewer outfalls was found to be largely local in nature because of the relatively small discharge from the individual sewers. However, objectionable conditions were noted in the vicinity of some of the more important outfalls.

To eliminate the objectionable pollution, the engineers in their reports to the City and Navy recommended the following basic procedures:

a. Collection of all Newport sewage into the main sewerage system.
b. Prevention of the escape of sewage from the Washington Square overflow except during storm flows.
c. Elimination of the Goat Island outfall.
d. Collection of all Navy sewage into a central sewerage system with treatment before discharge to the receiving waters.

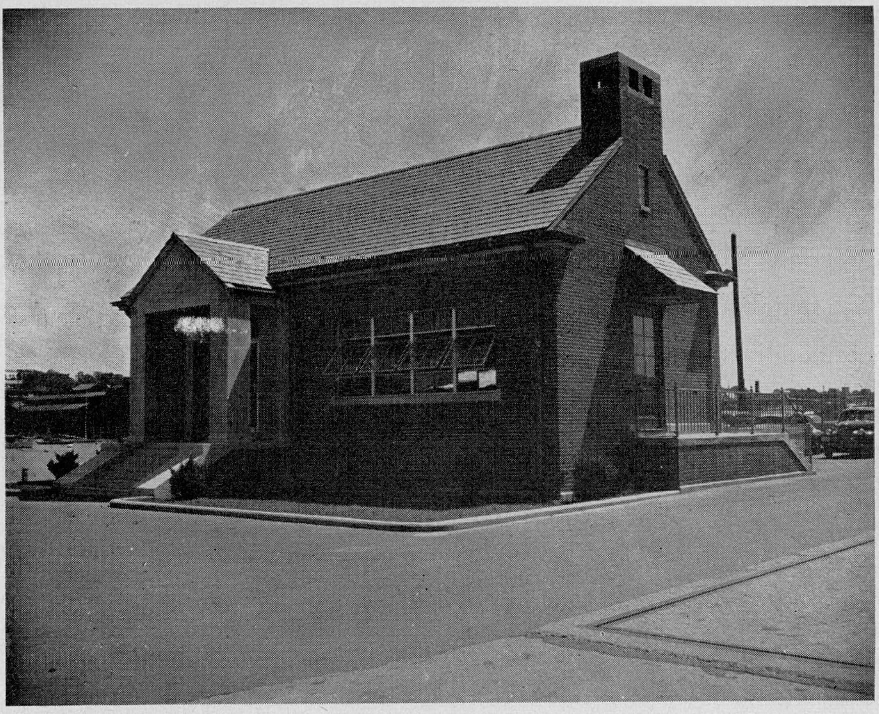
The interception of local sewers not discharging into the main sewerage system of the City was well covered in the engineers' 1928 report. In general, the 1948 report reiterated the previous recommendations and brought them up to date.

To prevent the overflow of sewerage at Washington Square during high tides, it was judged necessary to provide pumping facilities to lower the hydraulic gradient in the Thames Street trunk sewer. This project also necessitated reconstruction of the overflow facilities at Washington Square, and a tide gate structure at the Long Wharf overflow.

After a careful study of available sites for a main pumping station, the City Yard at Long Wharf was selected as the most suitable.

The main sewers were disconnected from the sewer leading to Briggs Wharf and the flows in them diverted to a new 72-in. conduit in Long Wharf. This conduit led to a point near the former overflow outlet at the upper end of Long Wharf. A weir in the conduit diverts all flows up to 2½ times the average dry-weather flow to a new 30-in. sewer in Long Wharf leading to a junction manhole opposite the main pumping station. Flows in excess of this amount pass over the weir and through a tide gate and discharge through the outlet to the harbor. The tide gate prevents the entrance of harbor water.

Sewage in the disconnected portion of the main sewer leading to Briggs Wharf was intercepted by a new 18-in. sewer in Washington Street, which in turn carried it to the junction manhole in front of the pumping station. A weir was provided in the main sewer which diverted up to 2½ times the dry-weather flow to the 18-in. intercepting sewer. Flows in excess of the above amount pass over the weir and are discharged through the existing overflow at Briggs Wharf. A



MAIN PUMPING STATION.

tide gate is provided below the weir to prevent entrance of harbor water. The general arrangement of these sewers is indicated in Fig. 3.

It was estimated that about 1,880 acres would be served by the main sewage pumping station. The station was designed for a maximum capacity equal to 2½ times the estimated dry-weather flow or about 8.2 mgd. The average rate of pumping was estimated to be about 3.5 mgd.

The sewage flows from the junction manhole through a 30-in. pipe to the pumping station. A sluice gate is provided in the station for shutting off the flow. The sewage passes through a mechanically cleaned bar rack with 1½-in.-wide clear openings for the removal of coarse solids and discharges to a wet well. The wet well is covered with removable, sectional, aluminum covers to permit access for the removal of grit with an eductor of the catchbasin cleaning type.

Three Worthington vertical-type, centrifugal, sewage pumps with variable-speed motors are provided in a pump room. Two of

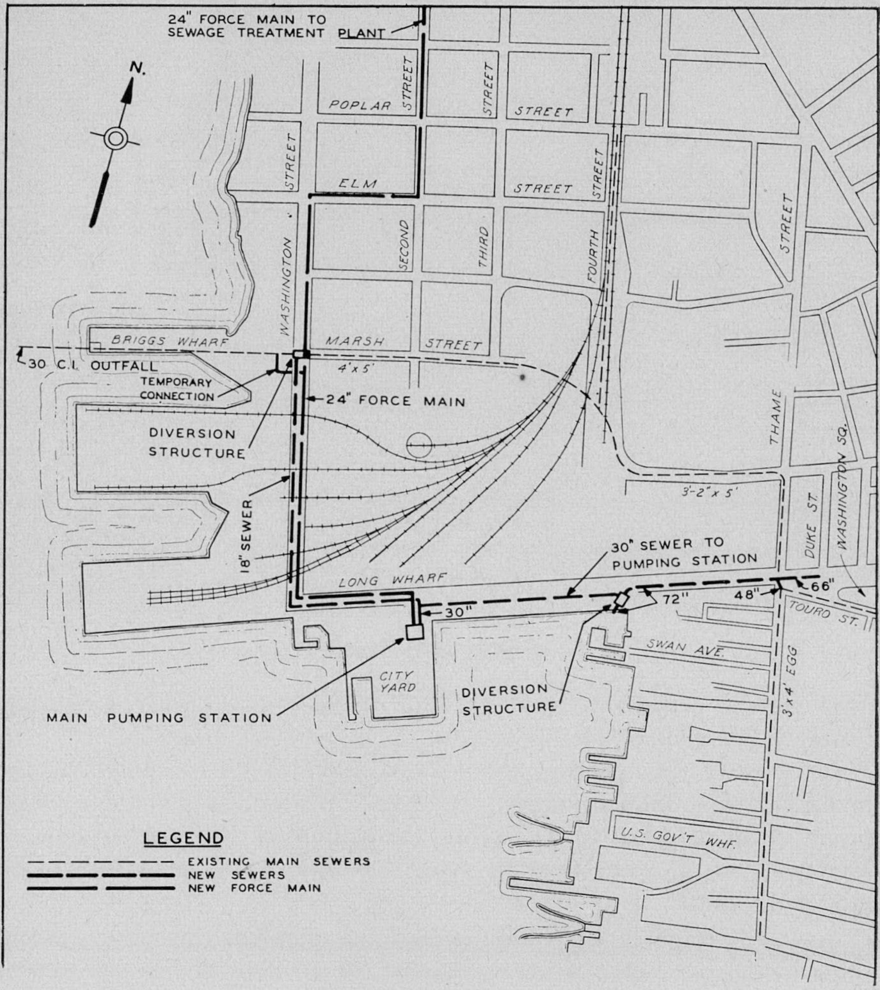
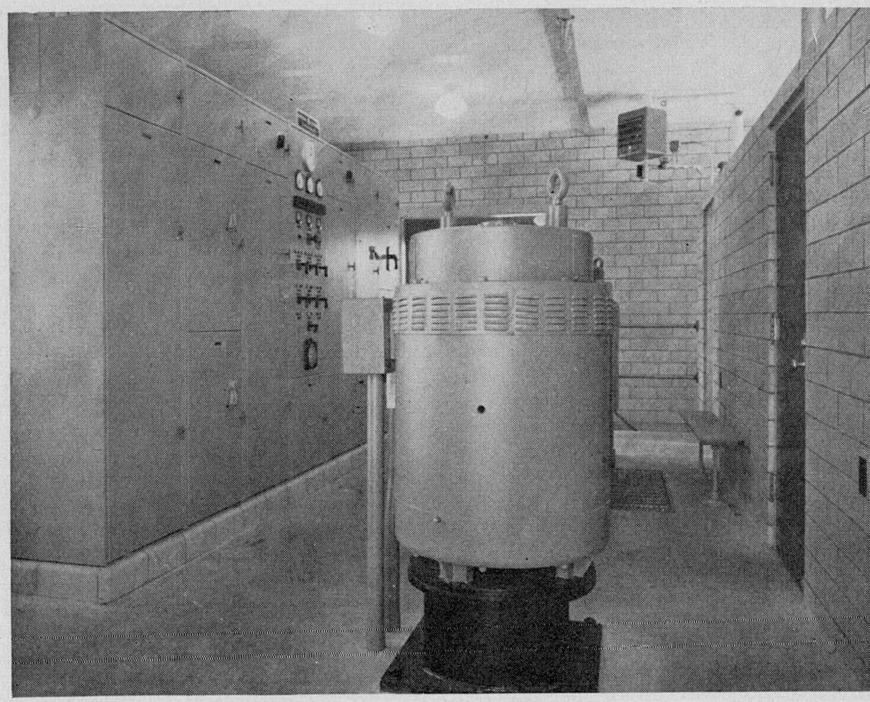


FIG. 3.—WASHINGTON STREET—LONG WHARF CONNECTING SEWERS.

the pumps operating together at full speed have a combined capacity of about 8.2 mgd. The third pump serves a standby. The operation of the pumps is automatically controlled by float-operated switches



MOTOR ROOM FLOOR SHOWING AUTOMATIC CONTROL PANEL AND ONE OF 3-75 H.P. 440V MOTORS.

operating on the sewage level in the wet well. Details of the pumping station are indicated in Fig. 4.

A venturi-type meter with a flow indicating, recording, and totalizing instrument is provided to furnish a continuous record of the quantity of sewage pumped.

The pumping station discharges to a 24-in. force main which eventually will lead to a proposed sewage treatment plant. Because of delay in constructing the sewage treatment plant, it was decided to construct only the portion of the force main from the pumping station to the point of crossing the outfall sewer at Briggs Wharf, a dis-

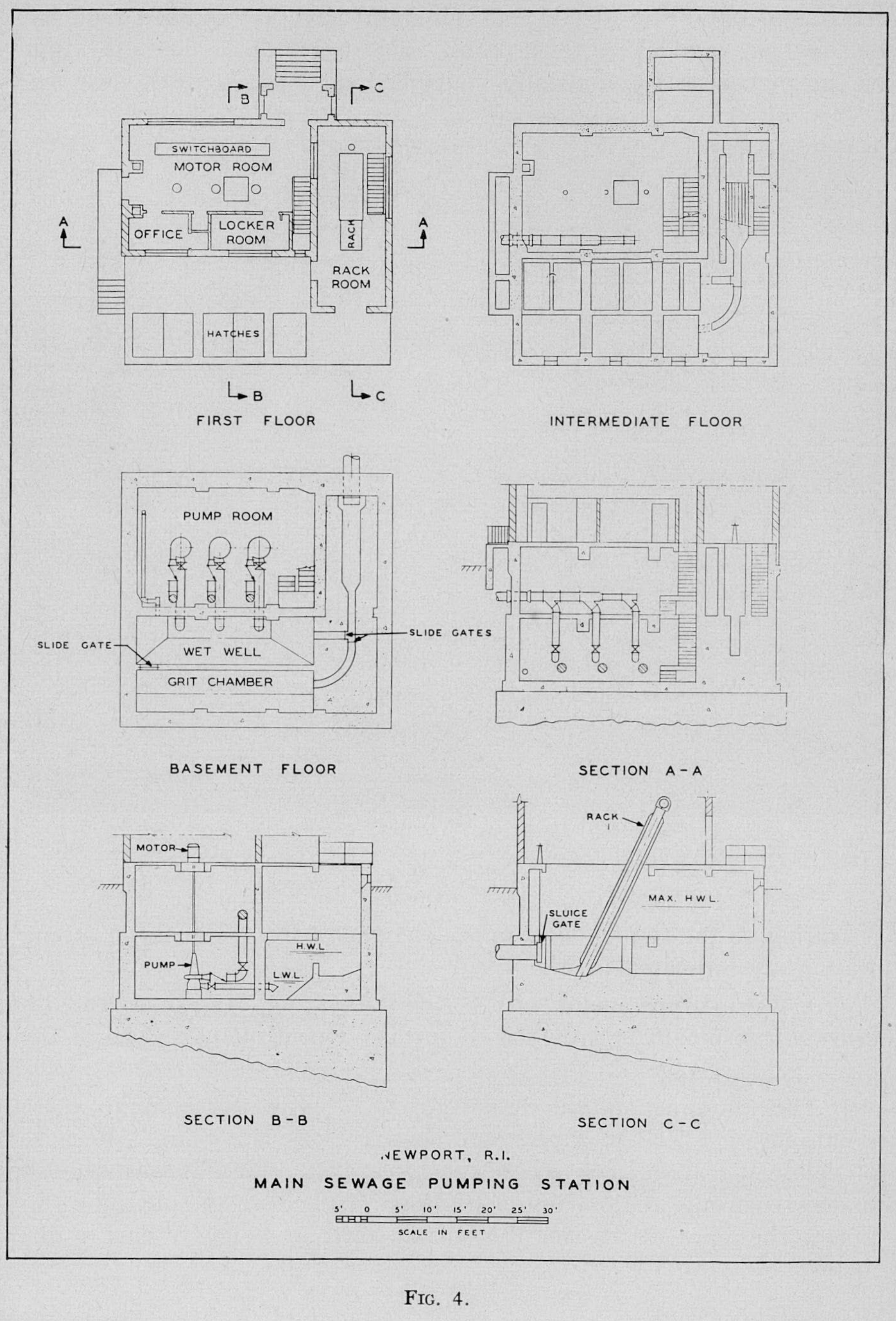
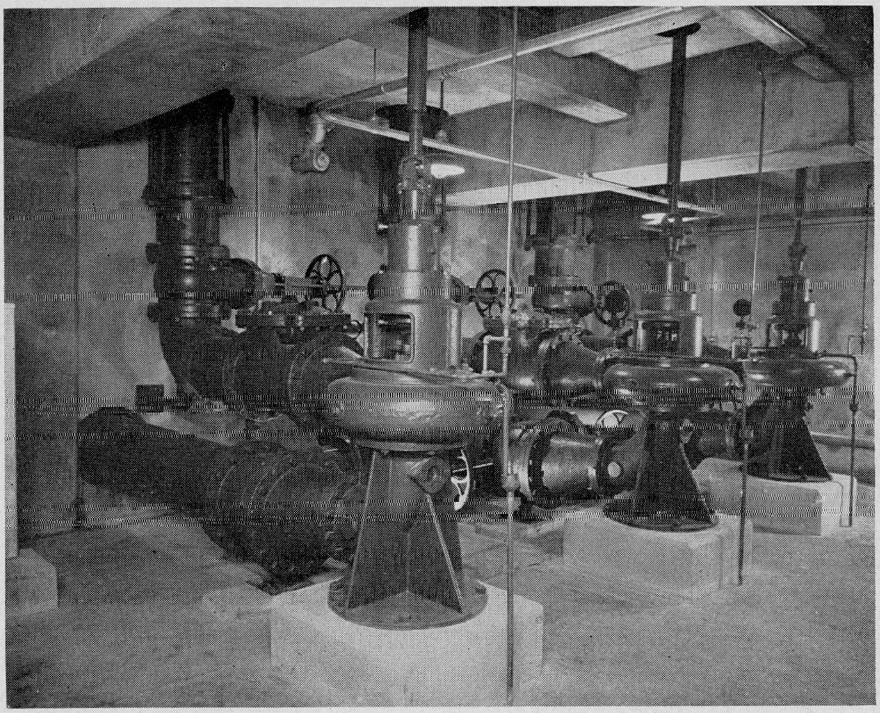

SWITCHBOARD
MOTOR ROOM
OFFICE
LOCKER ROOM
RACK
RACK ROOM
HATCHES
FIRST FLOOR
INTERMEDIATE FLOOR
PUMP ROOM
SLIDE GATE
WET WELL
GRIT CHAMBER
SLIDE GATES
BASEMENT FLOOR
SECTION A-A
MOTOR
PUMP
H.W.L.
L.W.L.
SECTION B-B
RACK
SLUICE GATE
MAX. H.W.L.
SECTION C-C
NEWPORT, R.I.
MAIN SEWAGE PUMPING STATION
SCALE IN FEET

Fig. 4.

tance of about 1,140 ft. A connection was made to the existing outfall sewer and the sewage discharged through the outfall. This arrangement permitted the early construction of the main pumping station to relieve flooding at Washington Square and thereby reduce the



PUMP ROOM.

nuisance of overflowing sewage at the Long Wharf overflow. This section of the force main and the main pumping station were placed in service in December 1953, and provided immediate improvement of the waters in the vicinity of Long Wharf.

The main pumping station was constructed by the Theodore Loranger Company of New Bedford, Massachusetts. The intercepting sewers and first section of the force main were constructed by T. W. Kidd of Fall River, Massachusetts.

To eliminate the pollution caused by the discharge of the main outfall, consideration was given to relocating the point of discharge.

Diagrams showing the direction of tidal currents for successive stages of the tide are presented on Fig. 2. These diagrams reveal that under certain tidal conditions currents sweep around either end of Goat Island and into the inner harbor.

Fifty-six per cent of the winds in the area are from a westerly direction, exerting force which carries untreated sewage discharged at the mouth of Newport Harbor toward the Newport waterfront. It is probable that untreated sewage discharged into the waters at any point east of Rose Island would under favorable conditions of wind and tide find its way into the inner harbor and Brenton Cove.

By extending the outfall sewer westward to the 10-fathom line beyond Rose Island, deeper submergence of the outfall could have been obtained, and it was felt that tidal currents would carry little if any sewage from such location back to the inner harbor. However, the outfall sewer had been broken about ten times since it was constructed and the last break occurred several years prior to 1949 and is still unrepaired. The shallow depth of water covering the outfall sewer, its lack of earth cover, the inflexible nature of its joints, and the number of vessels operating in its vicinity made it highly susceptible to rupture. Each break in the sewer caused a major portion of the sewage to be discharged directly in the inner harbor during the period of the break. Extension of the existing outfall sewer would have continued dependence upon this old pipe.

Replacement of the existing outfall sewer with new pipe along the present route would have been difficult because of the congested development on Goat Island. It was also believed that objections would be made by the Navy to such reconstruction because of interference with existing and possible future structures on Goat Island.

The best alternate location for an outfall sewer starting from Briggs Wharf would have been in a northwest direction terminating in the area northwest of Goat Island. Nearly 6,000 ft. of subaqueous pipe would have been required at an estimated cost in excess of $700,000.

The areas immediately west and northwest of Goat Island are convenient anchorage areas for naval vessels. Any extension of the outfall sewer to this area would have caused the City's sewage to be discharged into these anchorages, with resulting hazard to personnel on shipboard.

It was felt that properly disinfected sewage might possibly have been discharged without serious objection in deep water at some point west of Goat Island. However, disinfection is usually accomplished by chlorination and chlorination of sewage which contains settleable solids is not considered adequate disinfection. The particles of sewage solids contain in their interiors multitudes of bacteria which the chlorine is unable to reach. Following discharge from an outfall the solids gradually disintegrate and the bacteria are released. Before disintegration the solids may travel long distances and in doing so they may transmit the bacteria to distant areas. Meanwhile the bacteria may be protected from natural enemies (present in the water) which would otherwise destroy them.

To provide positive effective chlorination of the sewage, it was considered necessary to remove the settleable solids by passage through a sedimentation tank. In brief, primary treatment of the sewage appeared to be required.

There was no vacant land in the area adjacent to Briggs Wharf available for a sewage treatment plant site. Consideration was given to locating a treatment plant on the wharf or in the adjacent Fleet Wharf Area of the Navy. Since, however, these areas contain the only wharf space in Newport with railroad facilities, Navy officials held that to release any of the space for a sewage treatment plant might be hurtful to national defense.

For the above reasons, the extension or replacement of the existing outfall sewer was eliminated from further consideration in the program of pollution abatement. The investigation then turned to searching for suitable sites for a sewage treatment plant elsewhere in the City.

The location of the treatment plant to the south of Washington Square would have involved constructing a force main through the congested heart of Newport. Further, it would have placed the plant in the high-class estate district which is one of the attractions of the City. In view of these obstacles, no further consideration was given to locations in that direction.

To the north of the City there was vacant land available in the vicinity of the City incinerator. Studies showed that a suitable sewage treatment plant site could be developed north of Admiral Kalbfus Road on the west side of J. T. Connel Highway between the highway and the railroad. City sewage would be pumped to this site and the treated sewage would be discharged by gravity through an effluent

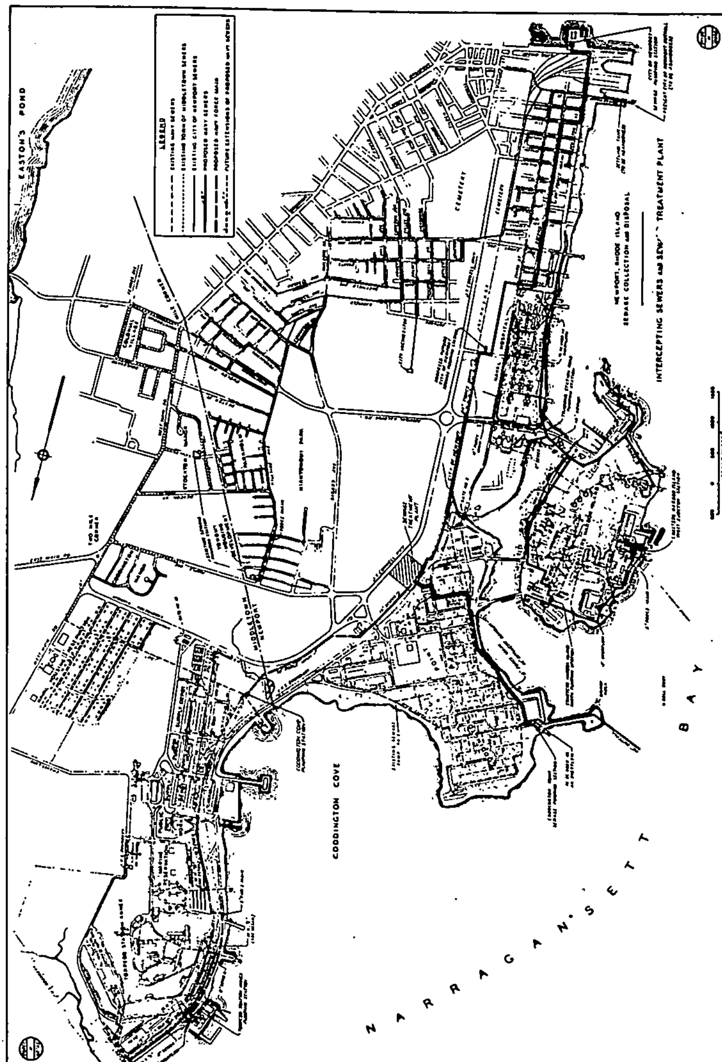


FIG. 5.—INTERCEPTING SEWERS AND SEWAGE TREATMENT PLANT.

conduit along the south side of Coddington Point and along a causeway leading to Bishop Rock. An outfall sewer about 500 ft. in length at Bishop Rock would discharge the effluent into about 40 ft. of water. Tidal currents reaching a maximum of 1.1 knots on the ebb tide and 0.5 knots on the flood tide sweep this location, thus assuring excellent dispersion of the sewage. The site consequently was adopted as the location of a central sewage treatment plant. The locations of the force main, sewage treatment plant, effluent conduit, and outfall sewer are shown in Fig. 5.

A 24-in. diameter force main was recommended leading from the main pumping station to the treatment plant. Its proposed route runs along Long Wharf to Washington Street, thence by Washington Street, Elm Street, Second Street, Sycamore Street, a proposed street parallel to Third Street and Dyer Street to the New Haven Railroad, thence by the railroad right-of way to the treatment plant. The total length of the force main is about 10,430 ft.

Since the unsewered area in the vicinity of the City incinerator could not be readily drained to the main pumping station, provision was made for a future pumping station to serve this area and to discharge to the new force main at Dyer Avenue.

The section of the force main from the main sewer at Briggs Wharf to the sewage treatment plant is now under construction by the R. Zoppo Co. of Norwood, Massachusetts, with completion scheduled in May of 1955. This contract also includes a bypass structure at the sewage treatment plant.

Before entering into a description of the sewage treatment plant, we wish to discuss the program for abatement of pollution caused by the Navy. A system of intercepting sewers, pumping stations, and force mains was recommended to intercept and collect the sewage discharged from the numerous outfalls in the Naval reservation and to convey this sewage to a central sewage treatment plant. Consideration was given to a separate treatment plant for the Navy at Coddington Point and for the treatment of Navy sewage in the Newport sewage treatment plant. Studies showed that a single plant to serve both the Navy and the City would be most economical to construct and operate.

The general arrangement of the proposed Navy intercepting system is indicated on Fig. 5. The plan adopted provides five sewage pumping stations and one pneumatic ejector station. The system is divided into three sections as follows:

a. The sewage from the system serving the Torpedo Station Annex will be collected at a pumping station and pumped to the outfall sewer serving the Marine Barracks. This outfall sewer will be intercepted by a 10-in. sewer and the combined flows carried to the Coddington Cove pumping station. Sewage from the Public Works and Naval Supply Depot areas will also discharge to this pumping station. A 30-in. combined sewer carries the sewage from the Naval Housing Area and a portion of Middletown and discharges it to Coddington Cove. This sewer has also been intercepted and discharged to the pumping station. Provision has been made to bypass storm flows past the pumping station and out through the old outfall. The pumping station discharges through a force main and a short stretch of gravity sewer directly to the central sewage treatment plant.

b. The sewerage system serving Coddington Point formerly discharged through an outfall at the tip of the point. The flow in this outfall sewer has been intercepted and carried to the Coddington Point pumping station. This station pumps the sewage through a long 12-in. force main to the central treatment plant.

c. Sewage on the west side of Coasters Harbor Island is collected at a pneumatic ejector station from whence it is discharged through a short force main and an intercepting sewer to Coasters Harbor North pumping station. This pumping station also receives sewage from the north end of the island. The sewage is pumped in turn to the Training Station Road pumping station which receives sewage from the south end of the island and from the Naval Hospital. The latter station pumps the sewage through a 12-in. force main in Training Station Road to the 24-in. City force main along the railroad right-of-way.

The design flows for the intercepting sewers and pumping stations were computed on a fixture unit basis. These were checked by an alternate basis which assumed a per-capita flow of 45 gpd. plus an infiltration rate of 2,000 gad. (gal. per acre per day) for the average flow, multiplying by a factor of 2½ to obtain the maximum flow.

The pumping stations were each designed to accommodate three pumping units. Two units operating together have a combined capacity equal to the expected maximum flow. Two units were installed initially with the third unit to be installed in the future, when required to meet wartime conditions. Manually cleaned bar racks are provided at each station.

An underground pneumatic ejector station was recommended for the Coasters Harbor Island West station because of the small expected flow.

Overflow structures were provided at each pumping station because the majority of the tributary sewers are combined sewers.

The systems serving the Naval Hospital, Coasters Harbor Island, and Coddington Point were designed by Metcalf & Eddy and are now under construction by R. Zoppo of Norwood, Massachusetts. The Coddington Cove pumping station was designed by the Navy Public Works staff and is generally similar to those designed by Metcalf & Eddy. This station and the force main were constructed by the A.T.R. Construction Co. of Newport, Rhode Island.

The quantity of Navy sewage will be measured by a venturi meter in the force main in Training Station Road and by separate Parshall flumes for the Coddington Point and Coddington Cove systems at the central treatment plant.

The quantity of sewage from the Newport system will be measured by a venturi meter in the force main at the main pumping station. A venturi meter will also be provided at the future Dyer Avenue pumping station.

The central sewage treatment plant will have a bypass structure which will contain the main plant bypass and the two Parshall flumes for measuring Navy sewage flows. From the bypass structure. the sewage will flow to a structure containing a main Parshall flume, two aerated grit chambers, and two 36-in. comminutors. From the comminutors the sewage will flow to sedimentation tanks, from which it will discharge to the effluent conduit.

The sedimentation tanks will consist of four covered tanks, each provided with a drag-type, longitudinal sludge collector and a screw-conveyor-type cross collector. Each tank also will be provided with a slotted pipe-type scum skimmer.

Sludge will be pumped to a pair of insulated digestion tanks arranged for stage operation. The primary tank will have a dome-type fixed roof and the contents will be heated by circulation through an external heat exchanger. Fixed nozzles discharging heated supernatant are to be provided for scum breaking. The secondary digestion tank will be provided with a floating cover. Gas will be collected and used for heating the primary digestion tank.

Digested sludge will be elutriated in a single-stage elutriation tank, and dewatered on a coil-spring vacuum filter. Sludge pumps, various auxiliary pumps, the vacuum filter, shop, office, laboratory, and personnel facilities will be located in a two-story main building.

The arrangement is designed to reduce outdoor operation to a minimum in the interest of personnel comfort and efficiency.

The dimensions and capacities of the various plant units and equipment are tabulated in the basic design data. The design population for initial construction is based on the civilian population of Newport (with allowance for a small amount of future increase) a pre-emergency population in the Naval facilities, plus allowances for tributary populations to be served in Middletown. The plant is designed to permit ready expansion to meet an estimated wartime load.

The results of analyses of sewage collected at Briggs Wharf in April 1949 indicated the City sewage to be relatively weak. The B.O.D. varied from 12 ppm. to 153 ppm.; suspended solids varied from 56 to 119 ppm.; and chlorides from 59 ppm. to 5,250 ppm. Only a small amount of industrial wastes is now discharged to the Newport sewerage system.

A contract for the construction of the sewage treatment plant was awarded to the D'Onofrio Construction Co. of New Haven, Connecticut, on October 25, 1954.

The work for the city of Newport was at first carried on under the direction of the Newport Sewer and Anti-Pollution Commission. This Commission was abolished at the beginning of 1954 when the City manager form of government was instituted. The work is presently under the direction of the Public Works Department and of Mr. William A. Gildea, the City Manager.

The Navy work has been carried forward under the direction of the First Naval District in Boston.

The Metcalf & Eddy engineering work was first directed by Mr. Frank Flood as partner-in-charge, with the senior author as project engineer on sewage disposal and Mr. John Raymond as the project engineer on drainage. Subsequently, the senior author has become partner-in-charge on sewage disposal and the junior author the project engineer.

THE ST. LAWRENCE INTERNATIONAL HYDROELECTRIC DEVELOPMENT

By William F. Uhl*, *Member*; Wilfred M. Hall**, *Member*; George R. Rich***, *Member*

(Presented at a meeting of the Boston Society of Civil Engineers, held on January 26, 1955.)

Authority for Development—Mr. Uhl

When the use of electricity began to grow shortly after the turn of the century, engineers realized the power possibilities of the St. Lawrence River. This mighty stream originates at the outlet of Lake Ontario, draining the largest body of fresh water in the world—the five Great Lakes. These lakes form a large natural regulating reservoir which smooths out the variations of flow from all tributary streams within the watershed and tends to equalize out-flow season to season. Below Lake Ontario the river drops 92 feet in less than fifty miles, through the International Rapids along the United States-Canadian border. The large steady flow combines with this fall to create an unusual natural opportunity for power production.

The necessary licenses and authority have been granted by the Canadian and United States governments to develop this power, and action agencies have been created to work together on a coordinated program.

The construction cost of the entire project is estimated at $561,730,000 and it will have an installed capacity of about 2,500,000 horsepower.

Authorization for the construction of this international power project stems from a number of enabling acts by international, national, and lesser political entities. The Boundary Waters Treaty of 1909 between the United States and Canada specified the uses to be made of international waters and established the International Joint Commission. Docket 68 of the International Joint Commission, approved October 29, 1952, constitutes approval of the applications of the two countries for the joint construction of the St. Lawrence Project, and stipulates the division of costs.

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**Vice President, Chas. T. Main, Inc., and Partner, Uhl, Hall & Rich.
***Director, Chas. T. Main, Inc., and Partner, Uhl, Hall & Rich.

Canadian legislation and Presidential order designated, respectively, The Hydro-Electric Power Commission of Ontario and the Power Authority of the State of New York as instrumentalities of each nation responsible for construction and operation of the project. A fifty-year license for construction and operation of the St. Lawrence Power Project by the Authority was issued by the Federal Power Commission in 1953.

Under New York State legislation the Authority has the right to employ the services of engineers, contractors, and others in the construction of the project and to issue revenue bonds and bond anticipation notes to finance its operations.

The St. Lawrence Power Project is a completely separate undertaking from the Seaway Project. It is based on separate authorizations, will be built under different contracting entities, and will be financed as an individual project. Joint use of the river by the two projects requires, however, that they be completely coördinated in planning and operation.

In the fall of 1953 the Authority engaged Sanderson & Porter

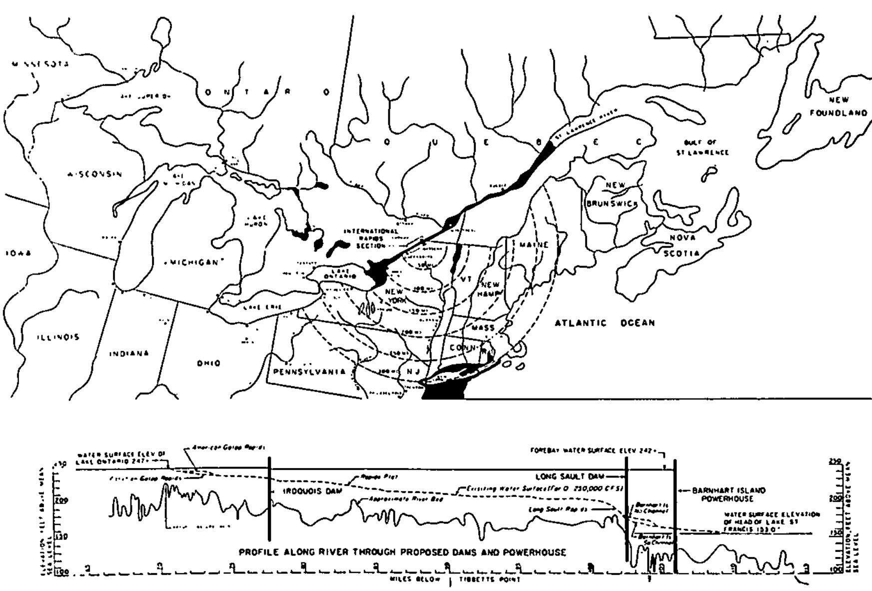

FIG 1 RIVER BASIN MAP

of New York and Uhl, Hall & Rich of Boston, a joint venture, as engineers to make an engineering report and estimate of cost of the project suitable for financing and budgetary control. This assignment was completed in August of 1954.

As of September 1, 1954, Uhl, Hall & Rich was engaged by the Authority to design, supervise all investigative field operations, manage the construction, and carry out initial operation of the St. Lawrence Power Project. Uhl, Hall & Rich is an affiliate of Chas. T. Main, Inc. The seven directors of Chas. T. Main, Inc. are all partners of Uhl, Hall & Rich and all seven partners are registered licensed engineers in the State of New York.

PROJECT DESIGN—MR. RICH

General. The St. Lawrence River Power Project will develop about two million kilowatts of hydroelectric power in the International Rapids Section, through which the St. Lawrence River drops about 92 feet in less than fifty miles, as shown by Fig. 1. Owing to the enormous storage capacity of the Great Lakes, the flow of the river is remarkably steady and over the 90-year period of record has a historical mean value of 236,000 cfs (See Figs. 2, 3, and 4). The maximum

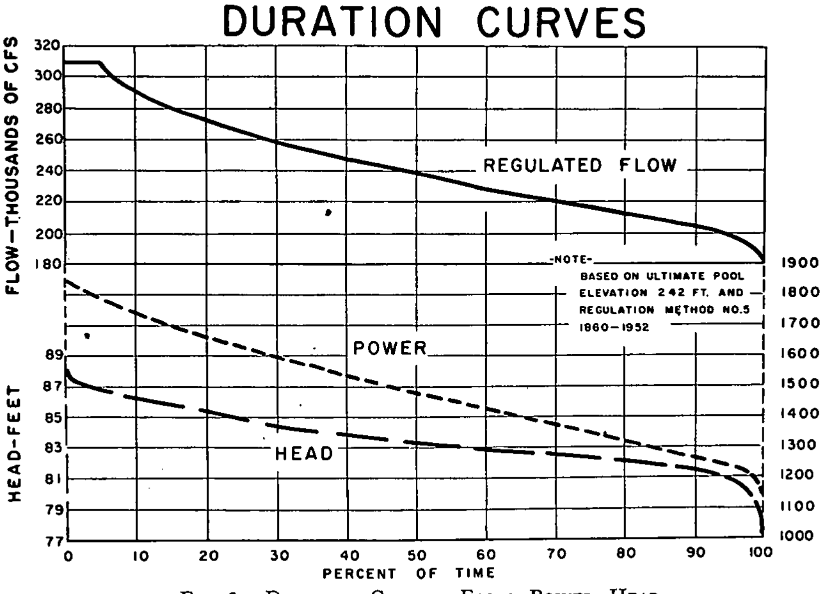

FIG. 2.—DURATION CURVES: FLOW, POWER, HEAD.

monthly mean flow is 314,000 cfs and the minimum monthly mean flow 144,000 cfs. The characteristic bedrock at the project site is dolomite interbedded with thin strata of shale, sandstone, limestone and gypsum. The dolomite rock appears entirely adequate to sustain the superimposed structures, although some grouting will be required to seal underground channels formed by dissolving gypsum.

POWERHOUSE TAILRACE RATING CURVE

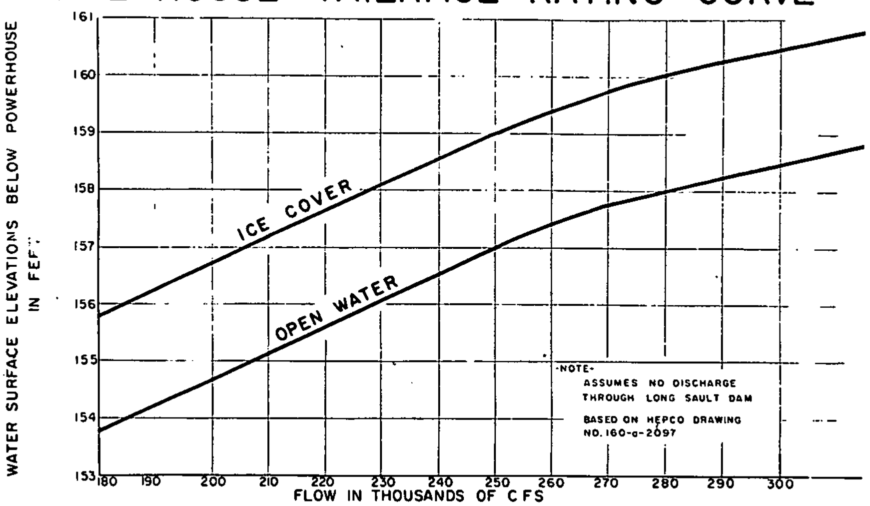

Fig. 3.

The major project features (See Fig. 5) comprise Barnhart Island Power Plant, Long Sault Dam, Iroquois Dam, channel improvements involving about 66 million yards of excavation and embankments containing about 10 million cubic yards of fill.

Barnhart Island Powerhouse. The Barnhart Island powerhouse is the largest single feature of the project, and in conjunction with Long Sault Dam and the attached dikes controls all channels of the St. Lawrence River in the vicinity of Barnhart Island and creates the pool for the development of power and the improvement of navigation in the International Rapids Section. The total installation comprises 32 generating units with a combined rated capacity of 1,824,000 kilowatts.

The powerhouse is a modified outdoor type, as shown in Fig. 6, in which elimination of the conventional superstructure affords an

RELATION BETWEEN WATER SURFACE ELEVATIONS AT LAKE ONTARIO AND POWERHOUSE FOREBAY

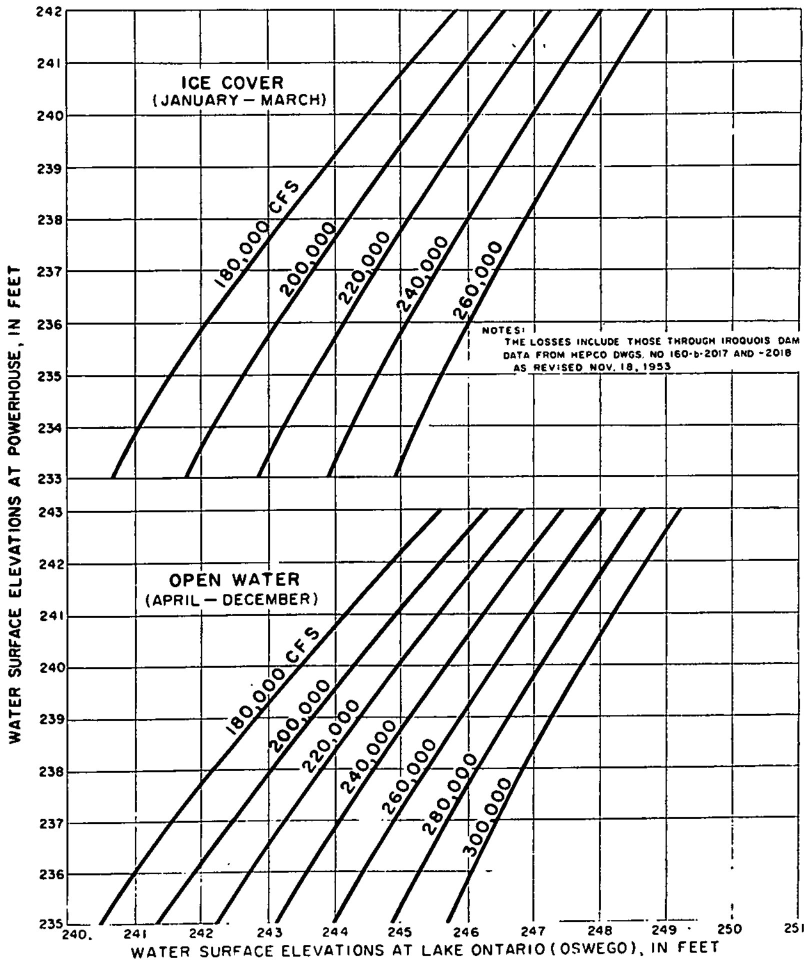

FIG. 4.

economy of about seven million dollars. All repairs to the generating units, except those requiring removal of the rotating elements, will be possible within the unit housing. For major repairs, rolling hatches in the deck above each generator will permit the removal of any part

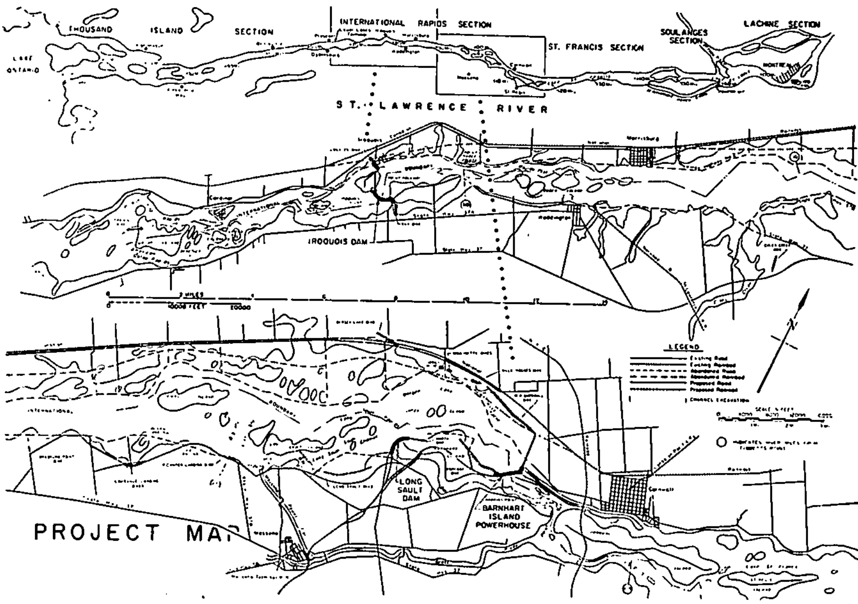

FIG. 5.—PROJECT MAP.

of the unit by an outside gantry crane traversing the length of the powerhouse. The traveling crane will carry a heated housing of sufficient capacity to inclose the part during its removal from the unit and during its transportation to the maintenance bays; the latter will be enclosed by appropriate superstructures of sufficient size to admit the gantry. Thus the process of both major and minor repairs can be fully shielded from the weather.

The powerhouse is a massive, reinforced concrete structure in which the combined mass of the intake and generating unit substructure is utilized to sustain the reservoir overturning load. The coördinated design of the intakes with the scroll cases and draft tubes will provide massive piers between adjacent units, extending through the powerhouse from forebay to tailrace, to carry the reservoir load to the foundation rock in a straight, unbroken line at right angles to the longitudinal centerline of the powerhouse. The heavy roof of the intake and scroll case serves as a horizontal girder, acting as the upper support for the intermediate intake piers. To accommodate the requirements of the construction progress schedule, the in-

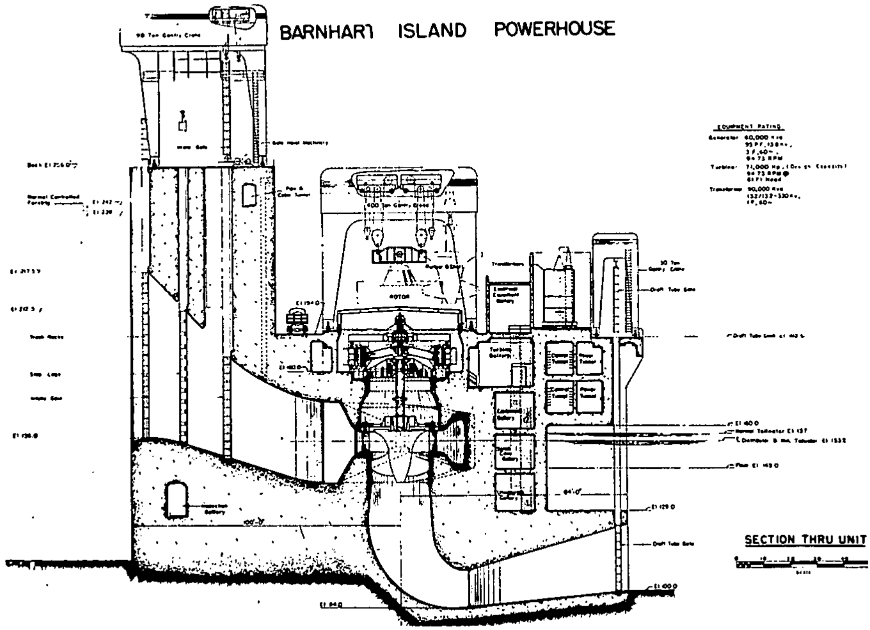

FIG. 6.—POWERHOUSE CROSS-SECTION.

take alone is separately stable for the condition of reservoir at Elevation 225.0.

Intake gates are provided for each turbine and are operated by means of individual fixed hoists. The gates are equipped with over-long, double-flanged roller bearing, wrought steel car wheels 24 inches in diameter, mounted upon forged-steel cantilever axles that extend through the end girders of the structural steel framing. The wheels bear upon 175-pound, Lorain-type crane rails, mounted upon the downstream side of the gate slot. The rails are mounted upon structural steel support towers embedded in the concrete of the intake piers to assure proper alignment. Sectional steel stoplogs and separate slots are provided to effect emergency closure. For handling intake gates after removal from the gate slots, and for handling stoplogs, trash rack sections and other miscellaneous equipment, two 90-ton electric traveling gantry cranes are provided.

The hydraulic turbines will be of the vertical shaft type with fixed-blade, propeller runners. The design capacity of each turbine

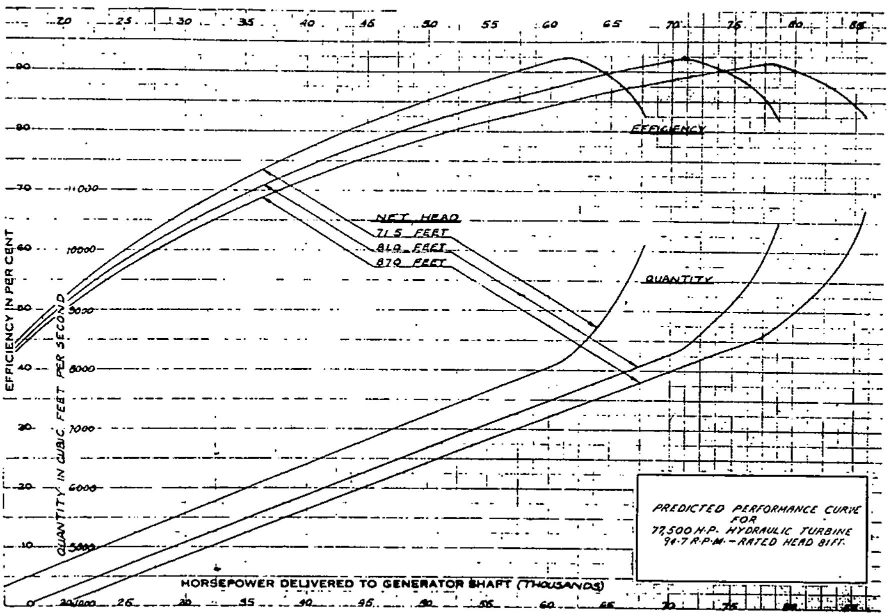

FIG. 7.—TURBINE PERFORMANCE CURVES.

has been established as 71,000 horsepower (Fig. 7) at the point of best efficiency, when operating under a net effective head of 81 feet, this being the head at which the units will operate the greater portion of that time during which the plant output would be near minimum capability (Fig. 8). The normal synchronous speed at which the units will operate is 94.73 revolutions per minute. The ultimate operating head will vary from 77.0 feet to 87.9 feet, as shown in Fig. 2, and the maximum capacity at full gate with the latter head will be 88,000 horsepower. Draft at best gate and 81-foot head will be about 8,400 cfs and maximum draft at this head will be 9,800 cfs. The runners will be about 240 inches in diameter and will be made of cast steel. The main turbine shafts will be 36- to 38-inch diameter forged steel, heat-treated, with suitable integrally-forged flanged couplings for connection to the generator shafts and runner hubs. The turbine guide bearings will be of the oil-lubricated type.

The generators are rated 60,000 kva, 57,000 kw, at 95 per cent power factor, and are capable of carrying continuously an overload equal to 15 per cent or 65,500 kw at 95 per cent power factor.

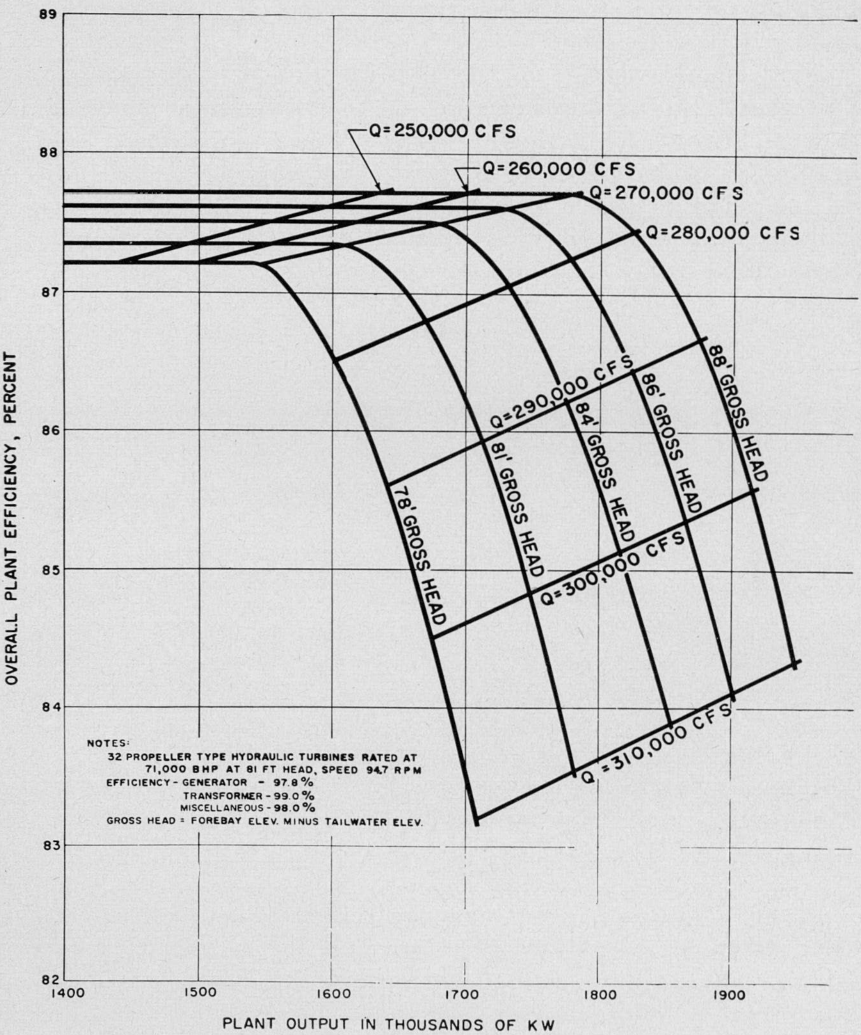

FIG. 8.—OVERALL PLANT EFFICIENCY.

A concentration of power such as that of the St. Lawrence Project requires immediate transformation to high voltage to avoid costly low voltage conduction and regulation problems. Consequently, the main step-up transformers will be located near the generators on the downstream platform above the draft tubes at Elevation 196. For the combinations of local and long-distance transmission that might obtain for this project, an initial step-up voltage of 230,000 volts was considered most desirable.

A distinctive feature of the electrical design is the elimination of overhead transmission structures on the powerhouse by virtue of the installation of 230 kv high pressure. oil-pipe type cables from the transformer banks to the switchyard. For this particular project, the superiority of cables from the standpoint of cleanness in design, maintenance, and reliability was found to outweigh any slight adverse differential in cost.

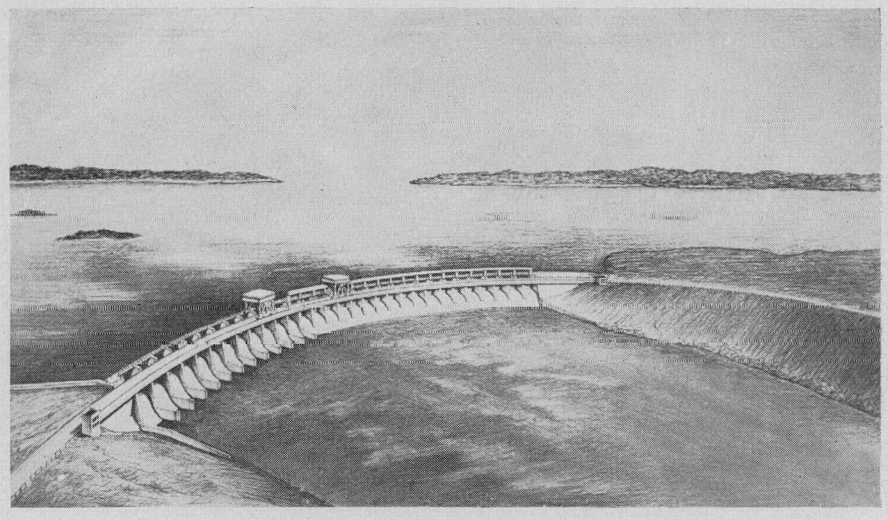

FIG. 9.—LONG SAULT DAM.

Long Sault Dam. The Long Sault Dam (Fig. 9) is a conventional, gated, ogee spillway (Fig. 10), curved in plan to conform with the natural location of the most favorable bedrock foundation with respect to both dam and cofferdam. The criterion for the gate capacity selected (30 openings 50 feet wide with crest at Elevation

217) was the ability to pass the maximum January flow (220,000 cfs) at the minimum pool elevation (Elev. 231). Eighteen of the gates, in the center of the spillway and opposite the deepest part of the river channel, are divided horizontally into two sections so that floe

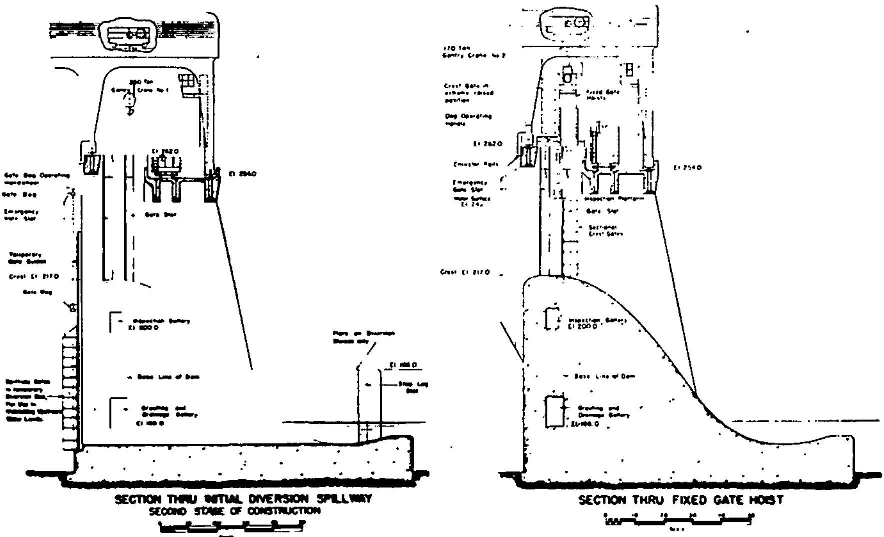

FIG. 10.—LONG SAULT DAM CROSS-SECTION.

ice may be skimmed with lower gate section crests at Elevations 229, 232, and 234. These 18 gates will have fixed individual hoists. The remaining 12 gates will be operated by either of two traveling gantry cranes on the deck. Owing to the severe icing conditions that exist in winter, special attention has been given to the question of keeping the gates and guides free of ice at all times. An air-bubbler system and adequate electric-heater elements will be provided for all guides, and the sectional gates will also have heaters.

To meet the requirements of the river diversion program, shown in skeleton form in Fig. 11, thirteen of the spillway bays will follow the intermediate stage of construction indicated by Fig. 12(a) while the remaining seventeen bays will be staged in accordance with Fig. 12(b).

Iroquois Dam. The Iroquois Dam is a buttressed gravity structure consisting principally of a series of piers and sluiceways. The

40 openings between piers are controlled by 32 two-section, fixed-roller, vertical lift gates and 8 single-section gates of the same type, all operated by two 225-ton traveling gantry cranes (Fig. 13).

The function of the Iroquois Dam is to act as a flow control and

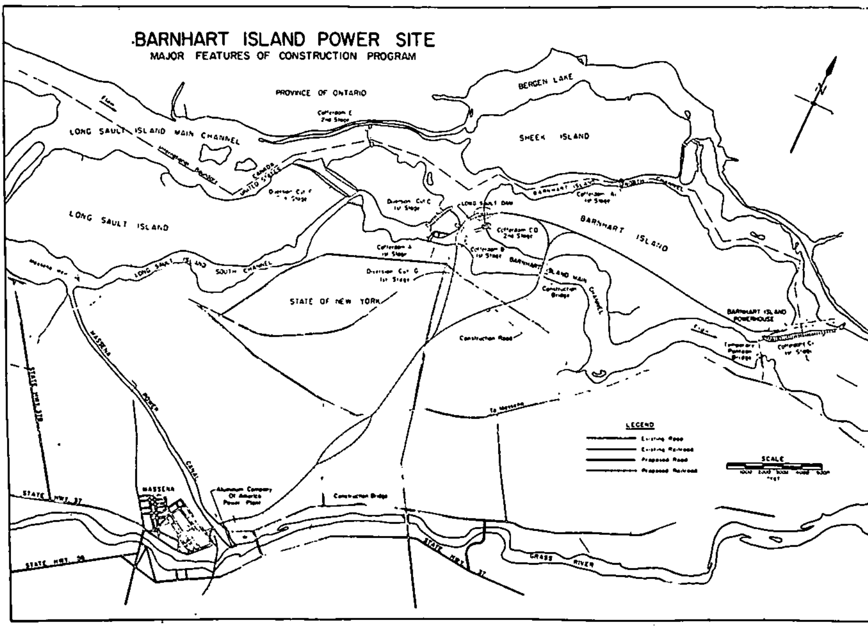

FIG. 11.—MAJOR CONSTRUCTION FEATURES.

regulator to replace the natural rock ledge or submerged weir control that presently exists near Galop's Island upstream from the dam, but which will have to be removed to provide the requisite channel for navigation. Iroquois Dam will regulate the outflow from Lake Ontario and restrict to the practicable minimum any variations in lake level that would affect adversely the tremendous investment in physical properties along the lake shore. It will also dampen seiches on Lake Ontario due to barometric changes and will expedite the formation of a very desirable solid ice sheet on the upper river. From the standpoint of power plant operation, Iroquois Dam will permit the essential operation of daily pondage without inducing resultant variations in the level of Lake Ontario. In the event of any major damage to Long Sault Dam or Barnhart Island power plant (by

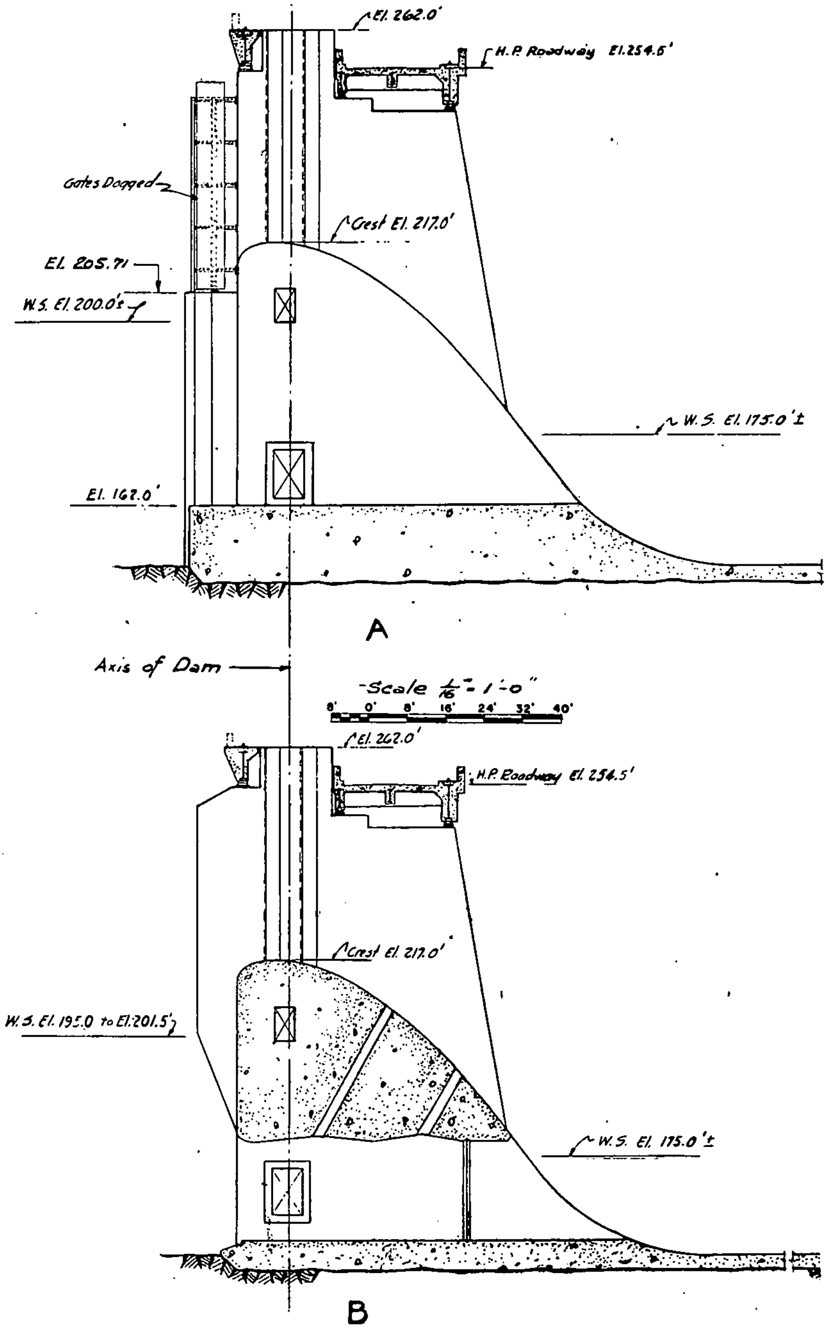

FIG. 12.—CONSTRUCTION SCHEME—LONG SAULT DAM.

bombing for example), Iroquois Dam would furnish an important second line of defense against the drainage of Lake Ontario (from which the natural rock submerged weir control has been removed). The gates of the dam will normally be fully open and the hydraulic head

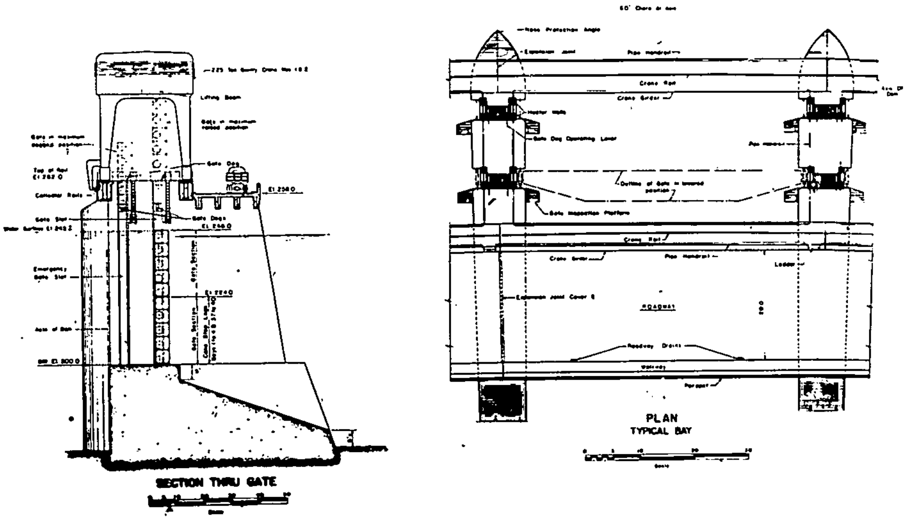

FIG. 13.—IROQUOIS DAM CROSS-SECTION.

loss through the structure will be in the order of only 0.2 to 0.3 feet.

Dikes. The pool behind Long Sault Dam and the powerhouse will rise to elevations higher than those in much of the surrounding areas of land and improvements. To confine the pool and to prevent outflow to contiguous drainage areas, a system of earth embankments will be constructed. These dikes, totaling about 14 miles in length, will have a maximum height of approximately 40 feet, and will include about 10,000,000 cubic yards of compacted embankment, protected by some 100,000 cubic yards of rock rip-rap on the resorvoir side.

The embankments will be constructed of earth materials, consisting mainly of glacial till, obtained from areas adjacent to the dike sites. The dike sections will generally be modified zones sections with impervious glacial till cores, supported by similar material compacted to a lesser extent and containing larger stones and boulders. Sand-and-gravel and rock sections will be provided in the downstream portions of the embankments to facilitate drainage and insure stability. The upstream slope will be protected by heavy rip-rap placed upon a filter blanket of sand and gravel; the downstream slope will be seeded to grass.

Compact glacial till covers much of the area where dikes will be built. It forms an excellent foundation material. There are, however, some areas underlain by soft marine clay deposits and in such locations the alignment of the embankments will be changed, if necessary. to reach typical, firm ground.

Channel Improvements. Channel excavation to be undertaken as a portion of the project will be of particular value to navigation by providing depth and restricting the velocity of the water. The channel work will also conserve power head and facilitate operation of the power project during the winter by limiting stream velocities so that a solid sheet of ice, rather than ice jams, may form over the forebay area. Channel work is required in three areas: above Iroquois Dam, where the natural rock weir control will be removed; below Iroquois Dam; and below the Barnhart Island Power Plant.

The channel enlargements may be classified in three general groups:

(1) Those required to produce a velocity not exceeding four feet per second at any time in order that navigation requirements be met: from Iroquois Point to Lotus Island and in the Galop Island reach.

(2) Those required to produce a velocity not exceeding 2.25 feet per second during the ice-forming period: from Morrisburg to Iroquois and from Iroquois Point to Lotus Island.

(3) Those economically justified for the conservation of power head: in the tailrace downstream from the powerhouse to Polly's Gut and in the Cornwall Island reach.

The usual backwater computations form the hydraulic basis for determination of the extent of channel enlargement required. Because of the complex nature of the branched river channel and the number of islands included, numerical computations were verified by extensive model tests at the hydraulic laboratory of the Hydro-Electric Power Commission of Ontario.

Project Construction—Mr. Hall

The engineering designs, plans and specifications described by Mr. Rich in the preceding paper are carried out in Boston. The large field force of Uhl, Hall & Rich, supplemented by contracts with survey and drilling companies, and by several New York State Departments for special services, is organized to carry on necessary field investigations, furnish construction management, and carry out initial operation for the project.

In addition to this direct work, a major coördination service is required. This applies to engineering plans, field investigations, construction management and initial operation, as related to corresponding activities on the part of Ontario Hydro, and to a lesser extent to relationship with the Canadian Department of Transport who are now operating the present 14-foot navigation channel and to the authorized agencies of the United States and Canadian governments, who are about to construct a new 27-foot seaway. Other numerous Boards and Agencies are involved in some phase of the over-all project to a still lesser extent.

Control Surveys. A series of monuments adjacent to the International Boundary from Cape Vincent to St. Regis were established by the International Water Commission, the International Boundary Commission, and the United States Lake Survey by previous surveys. A series of maps were prepared from these surveys and have been furnished to the Power Authority.

The Power Authority and Ontario Hydro have agreed upon a simple coördinate system of horizontal control and Ontario Hydro has computed latitudes and departures for all reference monuments, etc.

Vertical control has been established for many years from Cape Vincent to St. Regis by lines of precise levels carried out by the United States Lake Survey. These levels were re-run in 1941 and numerous ties were made to the latest monuments and bench marks.

A line of precise levels was run on the Canadian side of the river by the Canadian Department of Transport, and ties have been made between this and the United States Lake Survey line.

Aerial Surveys. The standard United States Geological Survey quadrangle sheets on the scale of 1:62500 and 1:25000 with 20′ contours were available for the whole area. Detail topographic sheets prepared by the United States Engineer Department, showing drill hole locations, test pits and river soundings, were included in their complete reports published in 1942.

Currently, aerial survey maps are being prepared under contract for land acquisition and other purposes related to the new reservoir levels. The aerial photography for these maps was carried out in the late fall when the foliage was off the trees and the ground bare of snow. These maps will be made on a scale of $1'' = 400'$ for an area of approximately 61,000 acres, or a scale of $1'' = 200'$ with $2'$ contour intervals for an area of approximately 6,700 acres.

Construction Schedule. The actual work on the project commenced in the summer of 1954 and several of the 16 generating units are expected to be in operation by the fall of 1958, with all of the units in operation before the end of 1959. In order to accomplish this, nearly all of the construction work must be completed in 1958 in order to raise the water level upstream from the powerhouse prior to the scheduled operation of the plant. All schedules have been carefully worked out in detail, from the acquisition of land to permit construction to start, to the final completion of the construction of the project. Practically all of the construction work contracts will be on a unit-price basis.

In order to proceed promptly with the work, a number of comparatively small construction contracts were awarded in 1954 without waiting for contract plans and specifications for the large major features. These contracts include a first-stage cofferdam for the Long Sault spillway dam, complete cofferdams for the Barnhart Island power plant (by Ontario Hydro), a pontoon one-season bridge to Barnhart Island (Fig. 14), a permanent bridge also to Barnhart Island, access roads and railroads, administration buildings, construction power lines and other items.

Construction contracts for the larger features of the project will be awarded early in 1955. There will be separate large contracts for the American and Canadian sections of the Barnhart Island power plant, and other large contracts will be for the construction of the Long Sault Dam, for the Iroquois Dam, and for several sections of the channel dredging. Numerous other features will be contracted for separately, such as earth dikes, highway and railroad relocations, rehabilitation, and miscellaneous other work.

Large items of permanent equipment will be contracted for separately. These include hydraulic turbines, generators, transformers, cranes and other items. These large items will be awarded early in 1955. Minor items of permanent equipment will be furnished by the various construction contractors involved.

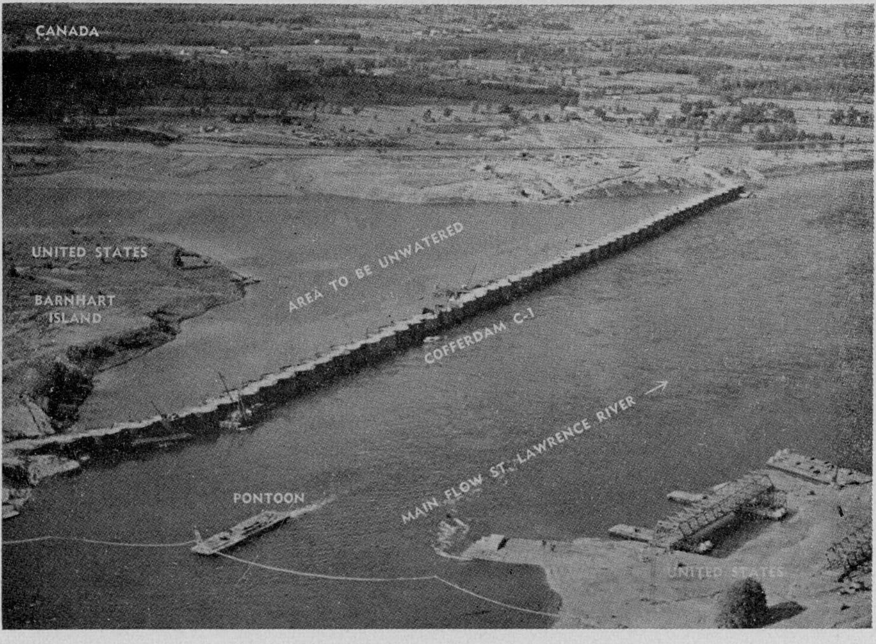

FIG. 14.—BARNHART ISLAND POWERPLANT SITE—MAJOR COFFERDAM C-1 AFTER CLOSURE; CELL FILLING NEARS COMPLETION. FIRST SUPERSTRUCTURE FOR TEMPORARY FLOATING BRIDGE TO BARNHART ISLAND IS BEING MOUNTED ON PONTOONS AT RIGHT, LATER TO BE FLOATED INTO POSITION BETWEEN ABUTMENT AND CENTER PONTOON IN MIDSTREAM.

Construction Plans. Access to the project is presently over existing secondary paved or improved gravel roads on both sides of the river. Access to Barnhart and other islands is by ferry. New access roads and a railroad extension are required for the construction of the large items of the project. The principal access road intersects the line of the United States section of the seaway and will eventually underpass the seaway. At that time the access railroad can be abandoned. The permanent bridge to Barnhart Island with a center span of 508 feet is a major structure, and for this reason the pontoon bridge was arranged for to give access during the year of 1955 and prior to the completion of the permanent bridge.

During the construction period all of the flow of the river will be diverted from the north side of Barnhart Island through the Long Sault Dam site south of Barnhart Island. This permits cofferdams for the power plant to be constructed at one time from the Island to the Canadian shore. Thus, construction work on the power plant can be carried on continuously until completion.

The Long Sault Dam will be constructed in two stages, the first section to be located inside of the cofferdam now under construction. The first-stage cofferdam extends out from the American shore to about the middle of the river, see Fig. 15. As the work inside of this

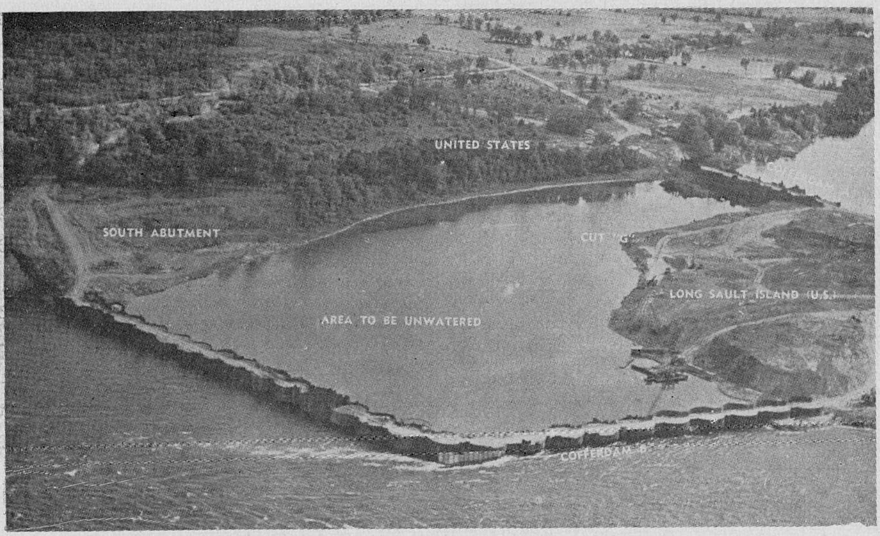

Fig. 15.—Long Sault Dam Site—Cofferdam B After Closure, Cell Filling Continues. South Abutment for Dam Excavated on Left and Cut "G" on Right.

cofferdam nears completion, a second cofferdam will be constructed, extending from near the middle of the river to Barnhart Island. Upon removal of the first-stage cofferdam, the river will flow entirely through openings left in the first section of the dam. Excavations are required on the Island upstream from the dam to divert water during the first as well as the second stage of construction.

The Iroquois Dam will be constructed in the same manner as the Long Sault Dam, and there will be two stages of cofferdams. The first section of the dam is largely constructed outside of the present river channel, and the first-stage cofferdam will be a comparatively minor structure. Thus, the second-stage cofferdam actually includes the larger portion of the river channel.

The control of the river must be carried out during all cofferdam stages and the raising of the pool without detrimental effect to the elevation of Lake Ontario upstream from the project and without detrimental effect to the elevation of Montreal Harbor downstream from the project.

Channel excavations are both within the natural river channel and outside the channel. Actually there are large quantities of both wet and dry excavation involved. This work is not continuous but is extensive within the limits of the project.

The disposal of the large quantity of material excavated from the channel is an important problem. There are a number of dikes and suitable material for these dikes can be secured from the seaway excavations when the latter are within a reasonable hauling distance of the structures. Thus the channel excavations, like embankments and seaway excavations will be scheduled and coördinated to make best use of the materials available.

The International Boundary intersects the powerhouse in the middle, intersects the Iroquois Dam, and numerous sections of channel excavation lie on both sides of the Boundary. The location of the International Boundary has a definite effect upon the construction plans for the project, the objective being to divide the work as well as the cost equally between the Power Authority and Ontario Hydro.

Construction Supervision. The construction and field engineers of Uhl, Hall & Rich will operate from an administrative area near the American shore of the river and the permanent bridge to Barnhart Island. Ontario Hydro will operate in a similar manner from an area on the Canadian shore north of the power plant. Both groups will have a number of sub-offices located at important points in the area from Iroquois Dam to the power plant. Laboratory work will be carried out in the field for the control of concrete in the large concrete structures, for the control of soil material for the earth dikes, and for the analysis of foundation material.

HYDRAULIC PERFORMANCE OF CHECK AND CONTROL VALVES

BY RONALD E. NECE* AND RICHARD E. DUBOIS**

(Presented at a meeting of the Hydraulics Section, B.S.C.E., held on February 2, 1955.)

INTRODUCTION

THIS paper is the outgrowth of a series of tests conducted at the Hydrodynamics Laboratory, M.I.T., on a series of check and control valves designed for specialized applications. These tests were conducted to determine the hydraulic performance of three types of valves: ball check, swing check, and diaphragm-type angle stop valves. The head loss was the major item investigated, and was determined for all valves. Observations were made of the stability of the moving parts of the check valves under differing flow conditions. Cavitation characteristics were not investigated because of the high pressure levels throughout the entire system for which these valves were designed.

The last portion is devoted to a correlation of head loss data for different types of commercial valves. This constitutes an attempt to bring together, in one place, a more comprehensive listing of the known hydraulic characteristics of certain types of control valves under various amounts of opening than is usually readily available to the hydraulic designing engineer.

The valves discussed here are all of the type which are most commonly found located in a pipe or conduit with the discharge from the valve remaining under pressure in the line, as opposed to those valves which most commonly serve as regulating devices at conduit outlets into the atmosphere.

TESTS ON SPECIAL APPLICATION VALVES

Scope of Testing Program

Descriptions of the valves tested are included below along with the test results obtained for each classification. The reasons for conducting the types of tests performed may best be evaluated by con-

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**Hydraulic Engineer, Capitol Engineering Corporation, Dillsbury, Pennsylvania.

sidering the performance requirements of these individual valves when installed in the system for which they were designed. A brief summary of these operating requirements follows.

The *check valves* must permit full flow in the normal direction only, closing when a flow reversal occurs. Since this is their sole function it is desirable that the head loss across these valves for forward flows be as small as possible. In contrast to commercial check valves, which have a fixed orientation in a stationary piping system, these valves may be installed so that they may be either horizontal or at large angles to the horizontal; proper performance is thus demanded through a wide range of valve inclination angles. Lack of motion of the moving components of the valve during steady state flow conditions through the valve must be considered; continuous motion indicates hydraulic instability caused by the flow through the valve and could in some cases lead to excessive abrasion due to continual contact between valve elements under this unstable, and usually noisy, mode of operation. The valves must close under low specified rates of reverse flow, but again to differentiate them from regular commercial valves a small amount of reverse flow leakage is stipulated.

The *stop valves*, which have as their primary purpose the closing off of a portion of a loop, may also be used as one means of regulating flows. Under normal operating conditions in the final prototype installation the flow may pass through these valves in either the forward or the reverse direction.

In line with the above, the following basic tests were performed. Other tests were also made in addition to those listed, but they were of a more specialized nature.

1. Steady State Forward Flow Head Loss:
   These tests were performed on all three types of valves. The check valves were tested horizontally as well as with the flow directed both upward and downward at an angle of 60° from the horizontal. (These positions shall be referred to as the +60° and —60° positions, respectively.) The angle stop valves possessed no moving components and were tested in the horizontal position only since their angle of installation would have no effect upon the flow characteristics.
2. Steady State Reverse Flow Head Loss:
   These tests were performed on the swing check valves and

angle stop valves. For the stop valves, these tests were identical in form to the forward flow tests. The moving clapper of the swing check valve was locked in place for these tests.

3. Torque Tests:
   These were conducted on one type of swing check valve in order to determine the variation of the hydrodynamic torque upon the valve clapper for varying geometries and discharges.
4. Stability Observations:
   Observations of valve component stability were made on the check valves. Some of the test valves were constructed of plastic or were provided with windows to facilitate observations; for valves having steel bodies, instability of the moving element of the valve usually gave audible indications.

Testing Equipment and Procedures

The test line for this program was constructed of 8-inch flanged steel pipe, galvanized inside and out. There were three loop configurations to allow for three basically different valve test positions. The term "test section" applies to that portion of the line in which pressure measurements were made.

1. Horizontal tests, check valves:
   This, the basic loop, had a straight, horizontal run of pipe 60 feet long comprising the test section; the test valve was located about 21 feet downstream from the upstream end of the test section. The check valves were fitted with flanges so that they could be interchanged in the opening provided for them in the line. This test loop configuration is detailed in Fig. 1. Pressure taps 1-5 provided the gradient of the approach flow; taps 17-22, more closely spaced downstream from the valve, furnished the gradient in the zone of pressure recovery; taps 28-33, extending to the end of the test section, gave the uniform gradient downstream from the valve.
2. Sixty-degree inclination tests, check valves:
   This loop was erected as shown in Fig. 1; also indicated are the two locations for the test valve, with flows either upwards or downwards with respect to the horizontal. With this loop in place the total center line length of the test section was increased to 70 feet.

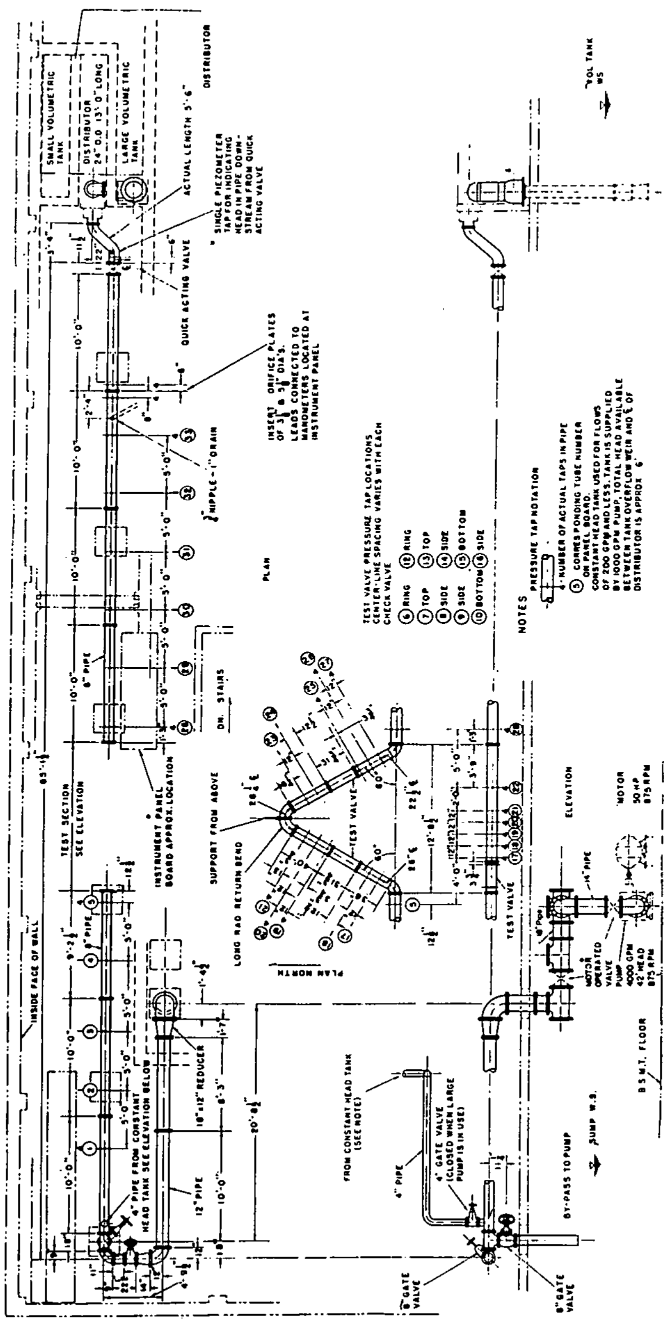


Fig. 1.—Details of Test Loop for Horizontal and 60° Inclination Tests.

3. Horizontal tests, angle stop valves:

   A complete change in the approach piping was required for this installation; the changed portion of the loop is shown in Fig. 2. The overall center-line length of the new test section, between pressure taps 2 and 33, became 63 feet. The angle stop valves tested were so fitted with flanges that they could be reversed in the line.

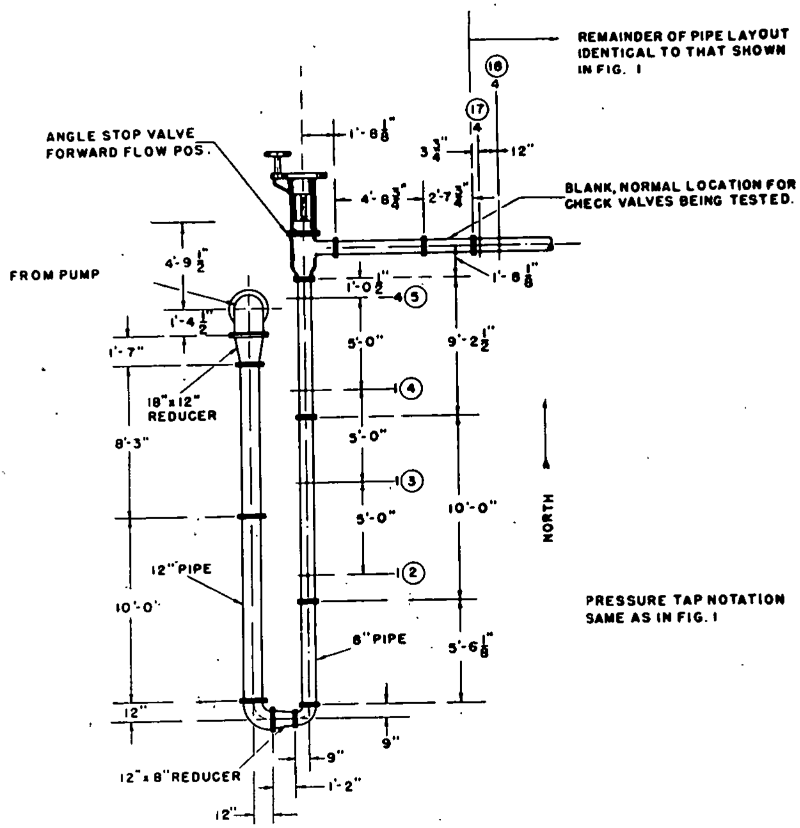


FIG. 2.—DETAILS OF ANGLE STOP VALVE TEST LOOP.

For the majority of tests the flow path through the system was from a centrifugal pump rated at 4,000 GPM at a 42-foot head, which pumped the flow from the storage tank and directly into the test loop; the discharge from the test loop could be directed back to the storage tank or into a volumetric tank. For early tests at low flow rates the 1,000-GPM centrifugal pump and constant head tank system indicated in Fig. 1 were used; this arrangement damped out pump pulsations. Later low flow tests were run using the 4,000-GPM pump, but with a change in metering techniques. A quick-acting gate valve located downstream from the test section served as the primary flow control; the pressure level at the upstream end of the test section was controlled by a by-pass valve through which water could be directed back to the supply pool before entering the test section.

The pressure connections on the 8-inch pipe consisted of either one ⅛-inch hole on the side of the pipe or a series of four interconnected holes evenly spaced around the periphery. Each pressure tap or tap group was connected by ¼-inch copper tubing to an individual piezometer column located on a centrally placed manometer panel. The levels in these columns, which were subjected to imposed air pressures for purposes of conserving space, indicated differences in head between pressure taps. An auxiliary manometer allowed actual heads to be determined throughout the test section.

Discharges were determined by calibrated orifice plates located in the 8-inch line downstream from the test section. Orifices of diameters 3⅛ and 5⅛ inches were used for flow ranges of 50-500 and 500-2,000 GPM, respectively. Head differentials were read on water-air and mercury-water differential manometers. For later tests involving even lower discharges a 2-inch orifice in a 4-inch pipe not shown in Fig. 1 was used; the entire flow was passed through this line, upstream from the test section.

Test Calibrations and Procedures

The experimental determination of the head losses through a valve requires that the gradients throughout the length of the test section be known for the condition when the test valve is not located within the loop. The horizontal and 60° loops were so calibrated, a piece of 8-inch pipe replacing the test valve. A number of tests allowed both the total friction loss across the section and the pipe friction gradient to be determined for any flow rate. No such calibration was made on the loop used in testing the angle stop valves; data from the horizontal line calibration were used to evaluate the pipe friction loss through the test section.

All friction calibration and valve head loss data were taken photographically; the 1/100-second exposure time provided instantaneous readings of the pressure taps, and was made necessary because of tube level fluctuations caused by the pump pulsations which could not be eliminated. Piezometer tube levels were read from enlarged photographs, values being estimated to the nearest 0.005 foot. Standard plotting methods, using distorted scales, were used to determine the valve loss; usually only the readings of taps 1-5 and 28-33 were plotted, since they were the only ones required to determine the non-recoverable loss through the valve. This procedure allowed the variation of head loss with flow to be found for each valve configuration. The results were correlated by means of the dimensionless head loss coefficient. K_L, defined by the expression for head loss in feet of fluid flowing, $h_L = K_L \frac{V^2}{2g}$.

Two methods, governed by valve construction, were used to make the torque tests on the swing check valves with circular clappers. One of these valves had a steel body and a non-counterweighted clapper. On this valve the clapper was locked in place by means of a lever keyed to the protruding shaft; the applied force was transmitted to a platform scale, allowing the torque required to hold the clapper stationary against a given flow rate to be determined. The second valve was constructed with a lucite body and a counterbalanced clapper; torque tests were made at lower flows and a torque wrench applied to the clapper shaft was used. No flow "tare" readings were taken for each valve at each setting tested with the valve filled with water and in place in the test loop.

Results of Testing Program

Three swing check valves were tested. Two of these were almost identical; valve "A" (see Fig. 3) was constructed of steel, and valve "M" was a plastice model. Clapper movement was restricted in valve "A" by means of a constant external torque applied externally to the clapper shaft; this function was fulfilled in valve "M" by an internal spring arrangement. Clapper diameter and closed position were identical in both valves; in valve "M," at intermediate angles of clapper opening, there was less clearance between the clapper and the valve housing.

In Figs. 4 and 5, the head loss and torque data are plotted in dimensionless form for each valve. In the expression used to define the torque coefficient K_T the torque is in pound-feet, ρ is the fluid density in slugs per cubic foot, and Q is the flow in cfs. In these

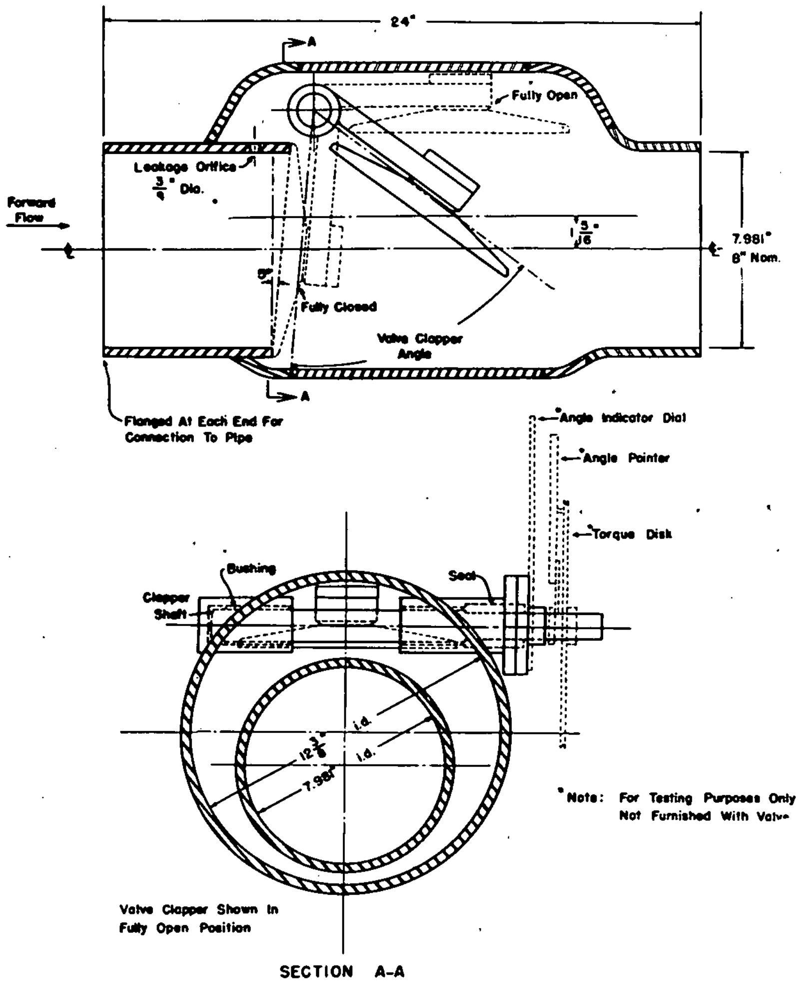


FIG. 3.—DETAILS OF SWING CHECK VALVE "A".

valves the head loss was due primarily to the sudden expansion downstream from the clapper; with fixed lines of flow separation around the clapper, the internal flow geometry was independent of Reynolds number except for extremely low flows and was fixed by the clapper angle. Therefore, K_L was a constant for any given angle, and because of the greater restriction of flow around the clapper in valve "M" the values of K_L are higher for this valve than for "A." The distribution of relative pressures on the two sides of the clapper was

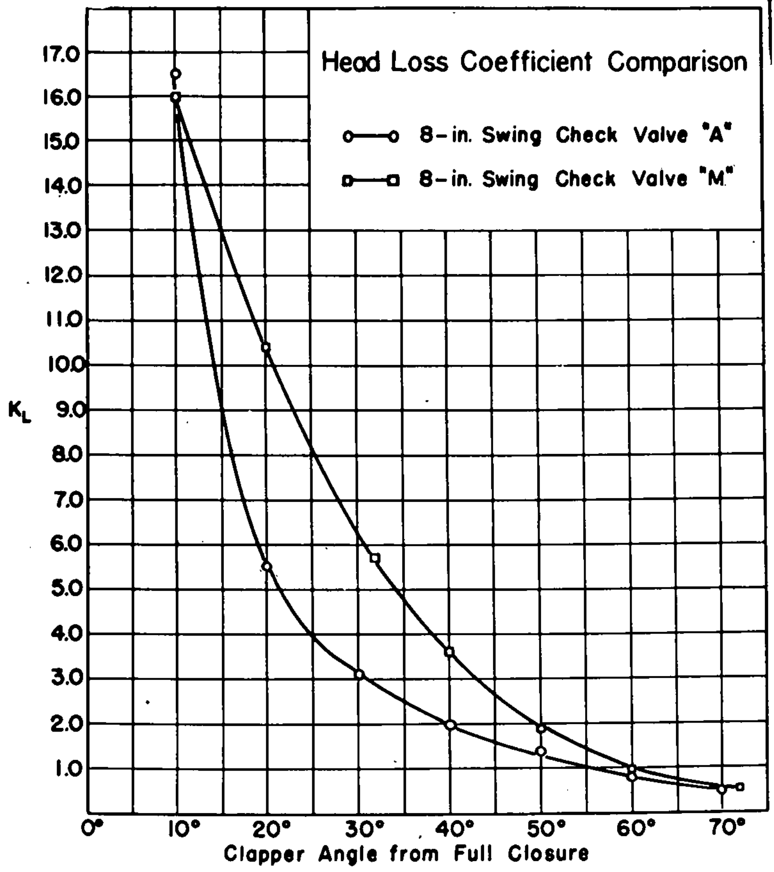


FIG. 4.—K_L vs. CLAPPER ANGLE, SWING CHECK VALVES "A" AND "M".

also fixed by the flow geometry, hence K_T is also a constant for a given angle. The two plots in Figs. 4 and 5 are plotted on the same basis to illustrate this similarity. There was no instability of the clapper in either valve; the lack of oscillation of the clapper showed that there were no large variations in separation eddies around the clapper.

Swing check valve "H," shown in Fig. 6, had a square flow duct with an eccentrically hinged neutral clapper which was balanced on the shaft. For flows from 200 to 2,000 GPM, the clapper had almost

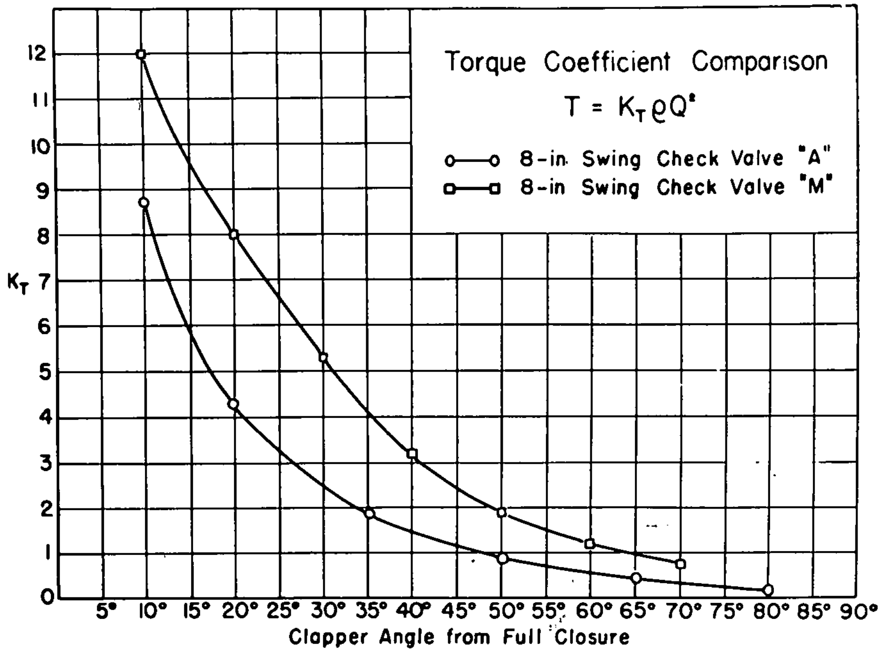


FIG. 5.—K_T VS. CLAPPER ANGLE, SWING CHECK VALVES "A" AND "M".

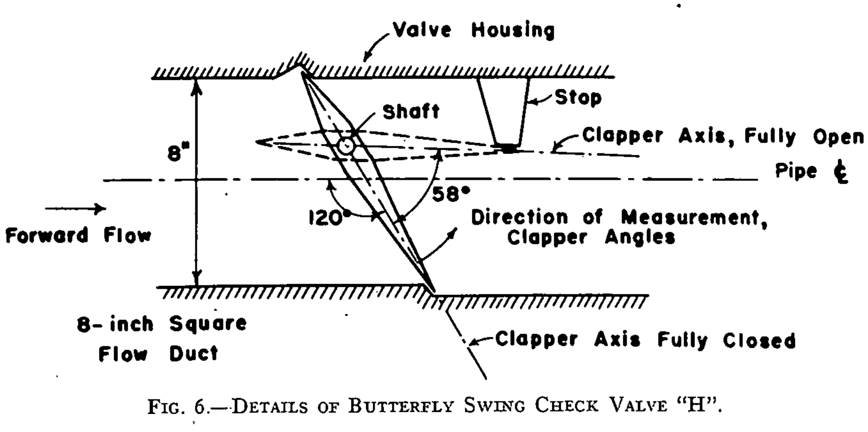


FIG. 6.—DETAILS OF BUTTERFLY SWING CHECK VALVE "H".

a constant equilibrium angle of 46° from full closure. This angle of the "airfoil" clapper was determined by its shape and the resultant flow geometry. The constant clapper opening angle indicated no changes in flow pattern caused by boundary layer shifts on the clapper surface.

All reverse flow head loss tests for the swing check valves were made with the clapper locked in place.

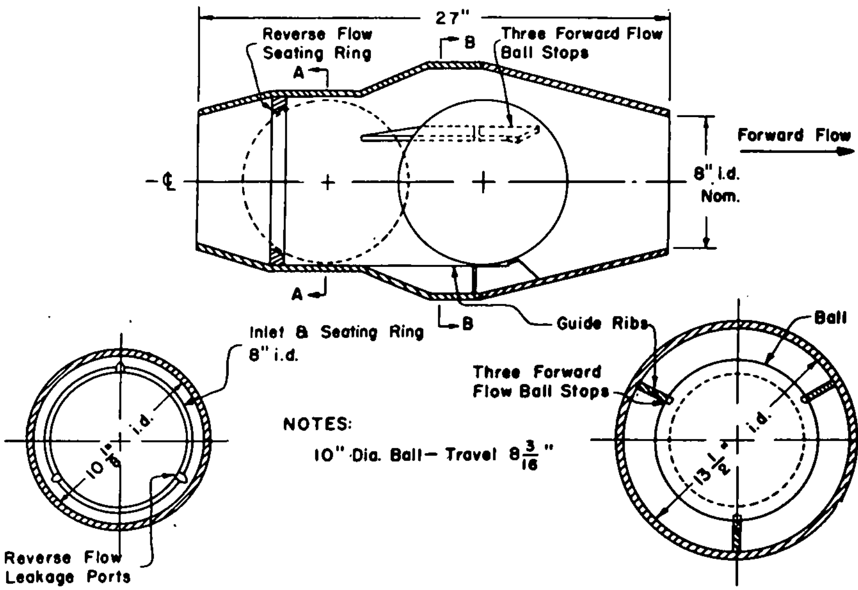


FIG. 7.—DETAILS OF BALL CHECK VALVE "B".

One of many ball check valve configurations tested is shown in Fig. 7. For this type of valve, stability of the freely moving ball was of more interest than the head loss. The stability problem was a large one for this valve because the ball was weighted so as to be almost weightless in water. Two types of instability could be expected: longitudinal instability, caused by the pressure distribution on the forward and back faces of the ball, preventing the ball from reaching a position of equilibrium against its stop at the downstream end of the valve; lateral instability, due to the inherent instability of flow around a sphere, leading to separation eddies and their resulting transverse forces on the downstream portions of the valve. To eliminate this latter, a 3-point forward flow support could be used, but this provides the danger of having the ball wedge in the forward position. The configuration of Fig. 7 was stable in all three test positions and had a low K_L of approximately 0.5. This value was higher at lower flows; the decrease in K_L could be explained by a transition from a laminar to a turbulent boundary layer on the ball, this change producing a smaller separation zone on the downstream side of the valve and hence a smaller flow expansion head loss.

The reverse flow rate needed to close the valve was an important specification, which at times was not satisfied while stability and head loss were. This was the case for the configuration shown in Fig. 7.

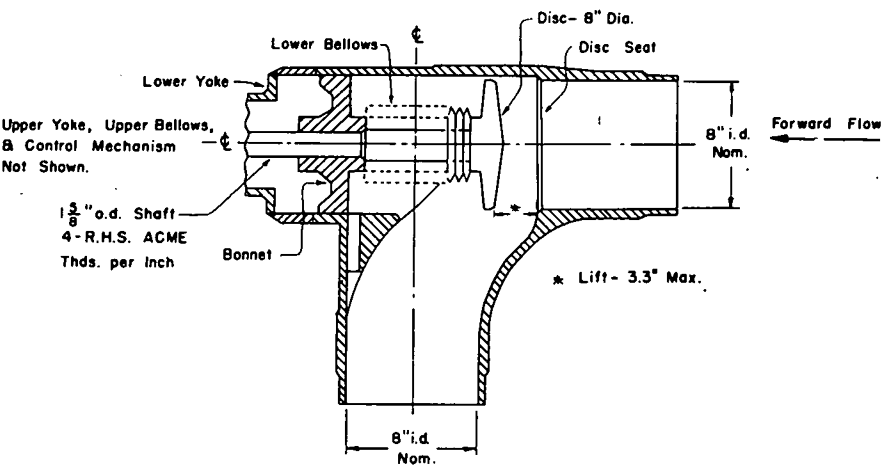


FIG. 8.—DETAILS OF ANGLE STOP VALVE "E".

Fig. 8 shows one of the angle stop valves tested. Valve "E" possessed more rounded flow contours than did valve "D," and for equal amounts of opening its K_L values were slightly lower. Despite the large apparent differences in the forward and reverse flow patterns through the valve, the forward and reverse flow K_L values were surprisingly similar.

CORRELATION OF CONTROL VALVE HEAD LOSS CHARACTERISTICS

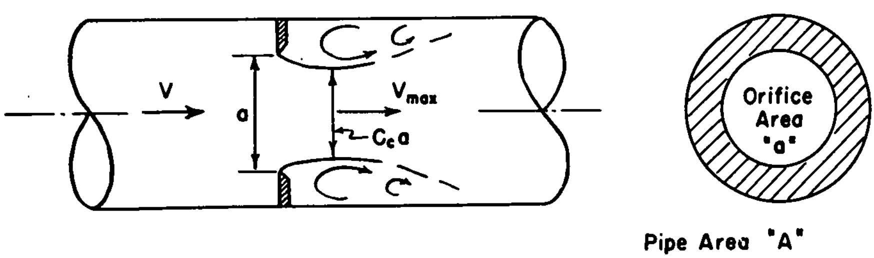
The only basis of comparison between these special-application valves and commercial valves which was to be investigated was the value of the head loss coefficient, K_L, for varying valve configurations. As none of the valves tested during this program are strictly similar to commercial valves, especially the two types of check valves, no obvious and direct correlation could be made between the data obtained on them and the information available on commercial units. The amount of design information about valves which is readily available is not overly plentiful. Therefore, no attempt at correlation could be made until a good basis of comparison could be obtained. This led to a literature search, which in turn led to a systematized grouping of past test data on commercial valves and an attempt to correlate these on a rational basis. The remainder of this paper is devoted primarily to the results of this study.

Test results were obtained from this literature survey for a number of different types of control valves. These were then grouped into two basic classifications for purposes of comparison, the basis of classification being the general flow pattern through the valve as determined by the shape and amount of opening. In the first category of valves are grouped those in which the flow through the valve proceeds generally in one direction with a minimum of deflection. Under this class are listed gate, butterfly, and plug valves; on the basis of the flow pattern, swing check and ball check valves could also be included in this category. In the second group are valves in which the flow path through the valve is very sinuous or in which the discharge from the valve is in a different direction than the inflow; under this classification are listed globe, angle stop, and Y-pattern valves.

The major cause of head loss in all of these control valves when they are being used to regulate the flow by throttling and are thus not fully open is the "Borda" loss due to the sudden expansion of the through stream as it emerges from the section of restricted area and discharges into the pipe or downstream portion of the valve. In the valves in the first category above, this contributes, with some exceptions which shall be discussed below, almost all of the total head loss. In the valves listed under the second classification, the velocities at the section of greatest area restriction are almost perpendicular to the inflow direction, through a cyplindrical surface included between the moving element, or disc, on one hand, and the disc seat on the other; this flow must then be recollected for final discharge from the downstream end of the valve. Because of these flow pattern differences, no common parameter was used to correlate K_L values for these two types of valves.

The dimensionless geometric parameter used for gate, plug, and butterfly valves was an area ratio. One of the two areas is the inside cross-sectional area of the pipe, based upon the nominal pipe size, and designated as A. The second area is the transverse projection of the restricted flow area of the valve, designated as a. For the gate and butterfly valves, and also the swing check valves, this is the projection of the visible through flow area; for the plug valve, it is the lens-shaped projection of the plug opening area as seen from one side of the valve. In all cases, the ratio a/A is indicative of the amount of valve opening.

The calculated head loss through a concentric circular orifice in a straight section of pipe was taken as the theoretical basis of comparison. The only head loss due to the orifice is here assumed to be that caused by the sudden expansion of the stream after the vena-contracta. This ignores all friction losses which occur both upstream and downstream from this point, but these are comparatively minor. For a known flow geometry, the expansion head loss can be obtained by use of the energy and momentum principles. Using the notation of Fig. 9, the head loss due to the orifice may then be written $h_L = \frac{(V_{max} - V)^2}{2g}$, where V is the approach velocity in the pipe, Q/A. The value of V_{max} depends upon the value of the coefficient of contraction of the orifice; for this study, average values of C_c obtained by Weisbach for different a/A ratios in water tests on concentric orifices were used. With C_c then known for a given a/A ratio (assuming



Flow Through a Circular Orifice

FIG. 9.

sufficiently high Reynolds numbers so that variations in discharge do not affect the flow geometry), continuity allows the head loss to be expressed as $h_L = K_L \frac{V^2}{2g}$, where K_L is now a function of the area ratio only. The rational value of K_L for valves under the first category was calculated on the basis of the a/A ratio, using the same C_c values as those for the same area ratio with a concentric circular orifice.

As stated, this area ratio is meaningless for valves in the second category. For these valves, a lift-diameter ratio was chosen as the dimensionless parameter indicative of the amount of valve opening; this is the ratio of the lift L of the disc above the seating ring to the diameter D of the seating ring. The use of this parameter did not lead to a rational or theoretical basis of comparison, but did provide a means of correlating the experimental data.

The remainder of this paper is a discussion of the collected head loss information, considered on these two bases. The M.I.T. test results are here discussed as merely another contributing source of data. The available information shall be discussed with respect to its agreement or disagreement with the expected values based upon the chosen geometric parameters. While the sudden expansion following the contraction area reduction in the valve accounts for most of the loss, losses are also due to changes in flow direction and to boundary resistance. These three items should then all be considered in an analysis, and the physical features of the valve which govern them: size and shape. Because of the varying physical shapes of the valves discussed, each is treated separately.

Gate Valves

Fig. 10 is a plot of K_L vs. a/A for a range of sizes of gate valves, with data from a number of observers. The area ratios can be easily converted into a more directly useful ratio of stem travel to valve diameter. All but one of the curves (number 14) shown on the figure are for conventional rising disc-type valves.

Practically all of the K_L values are larger than for the comparable concentric circular orifice with the same a/A value. For large a/A ratios, when the valve is almost fully open, there is relatively little contraction of the flow past the disc, but the boundary configuration is a source of flow disturbance. For small a/A ratios, the through-flow area is small, and the flow pattern is much like that through a tubular orifice; the stream contracts as it passes the up-

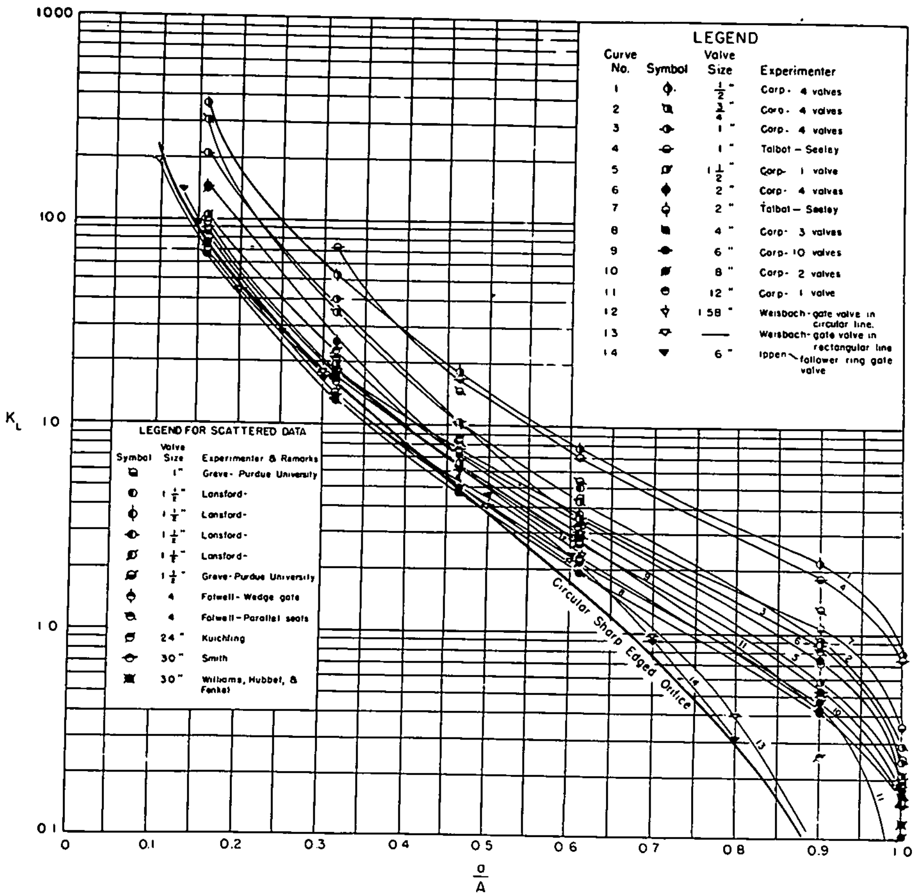


FIG. 10.—K_L VS. $\frac{a}{A}$, VARIOUS VALVES.

stream face of the disc, expands in the region between the disc and the valve housing, and then emerges as a jet below the downstream disc face. This produces a double expansion loss.

Scale effect is very pronounced. For the smaller diameter valves, the double expansion occurs at larger a/A ratios than for larger valves because of the relatively greater disc thickness. The boundary configuration and gate recess are also more effective on the flow pattern in the smaller valves. Some of the most common data for gate valves

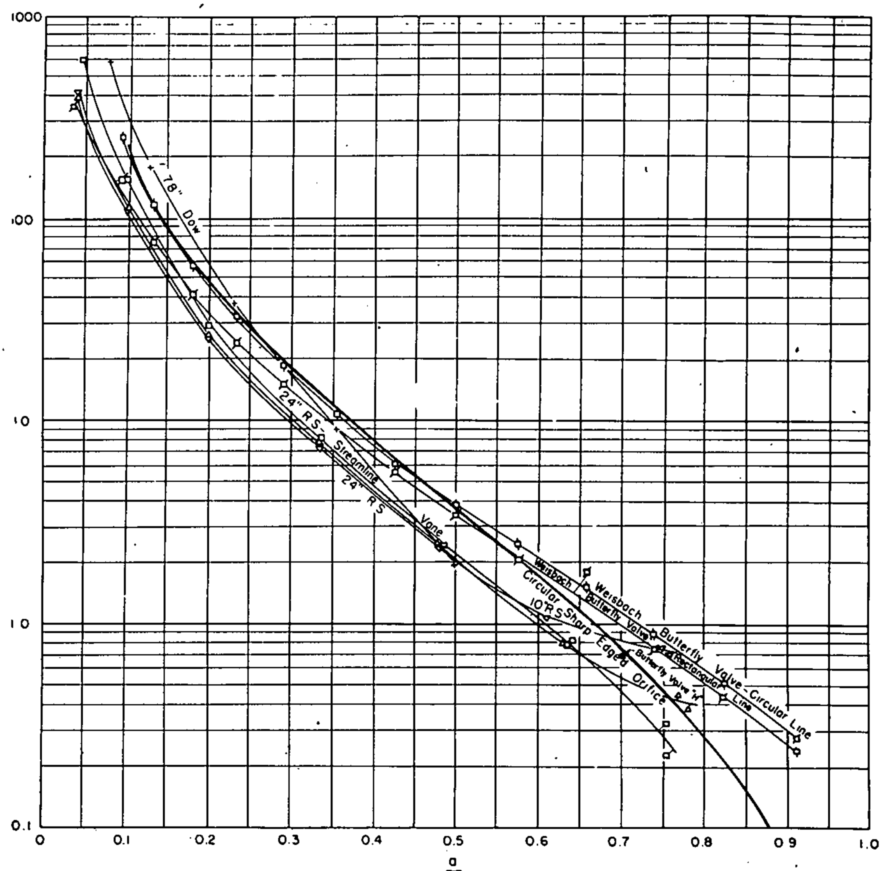


FIG. 11.—K_L vs. $\frac{a}{A}$, VARIOUS BUTTERFLY VALVES.

which are readily available, and in which scale effect is not mentioned, evidently apply correctly to small valves of nominal diameter range of 1 and 2 inches.

Curves 9 and 14 of Fig. 10 show the difference in head loss through a ring-follower gate valve and a conventional valve of the same nominal diameter and with a traveling gate disc. The projected area a for the ring-follower valve is lens shaped, and for the conventional type it is crescent shaped; the simpler through flow area shape gives a smaller K_L for the former.

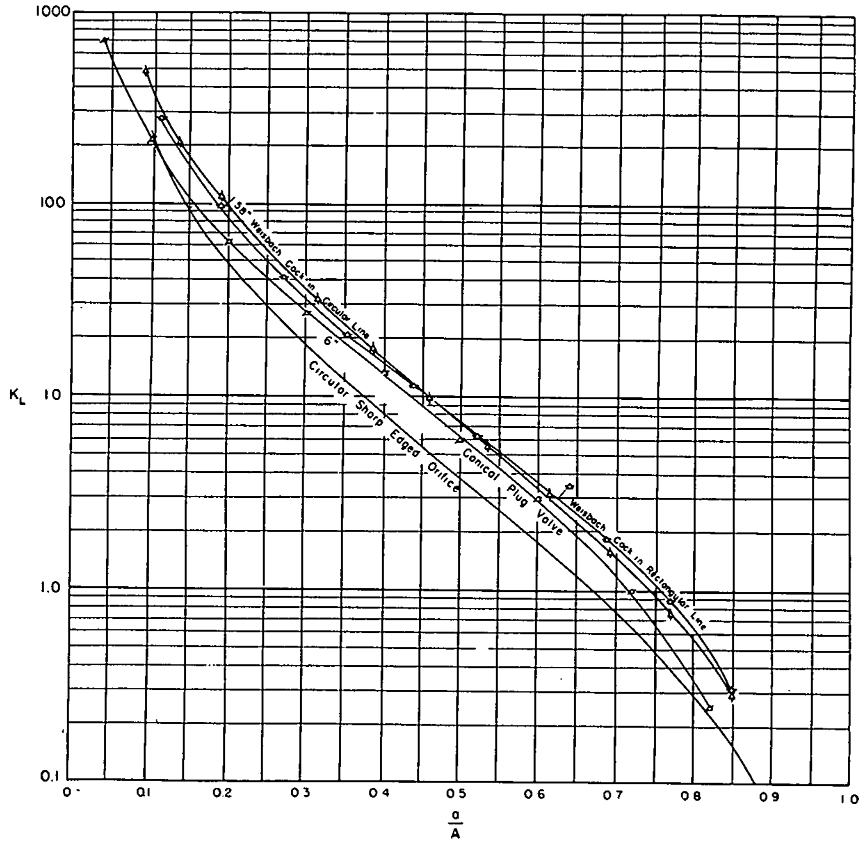
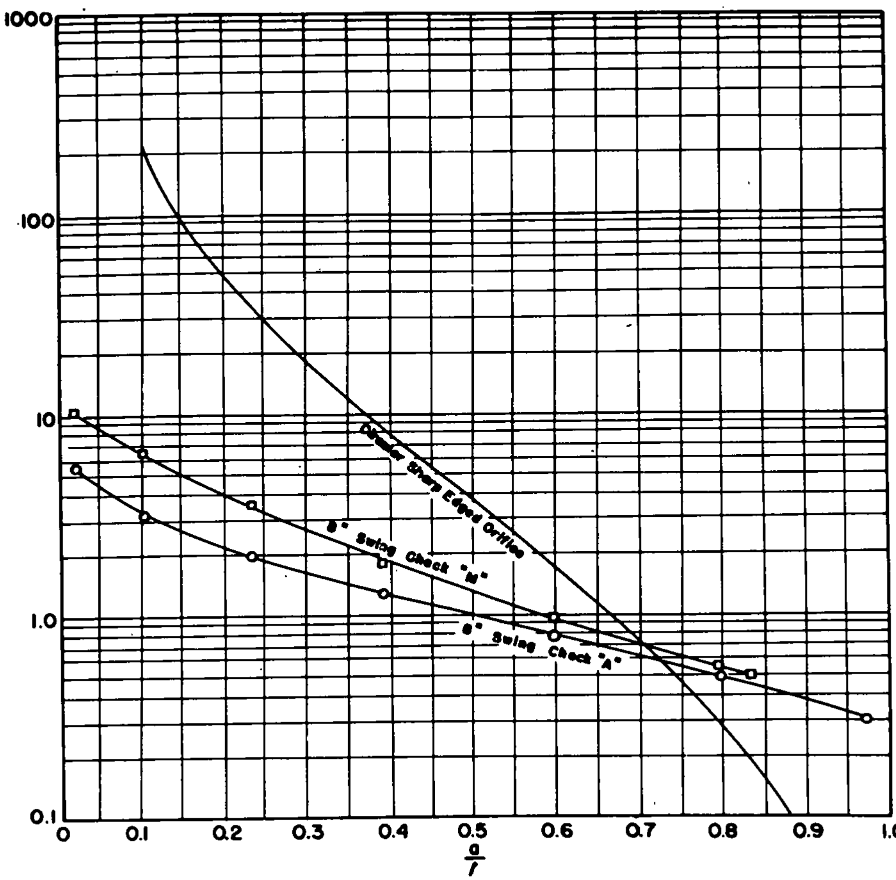


FIG. 12.—K_L vs. $\frac{a}{A}$, VARIOUS PLUG VALVES.

Butterfly Valves

The K_L values for butterfly valves, data for which are shown in Fig. 11, are generally smaller than for a concentric circular orifice of equal a/A ratio. This is due primarily to the smaller contraction resulting around the leading edge of the butterfly valve than at the orifice. Variations in K_L are primarily a function of C_c of the flow past the disc, and this in turn is largely a function of the degree of streamlining of the disc edges.



K_L vs. $\frac{a}{A}$, Swing Check Valves "A" and "M"

FIG. 13.

Valve "H," actually a swing check valve, is best analyzed here because of its flow pattern similarity to that of a butterfly valve. The single point representing the equilibrium angle of the valve clapper is almost coincident with the circular orifice curve.

Plug Valves

Fig. 12 gives data for this type of valve. For equal a/A ratios, K_L values rise faster for plug than for gate valves. A double contraction and expansion of the flow path occurs at lower a/A ratios than

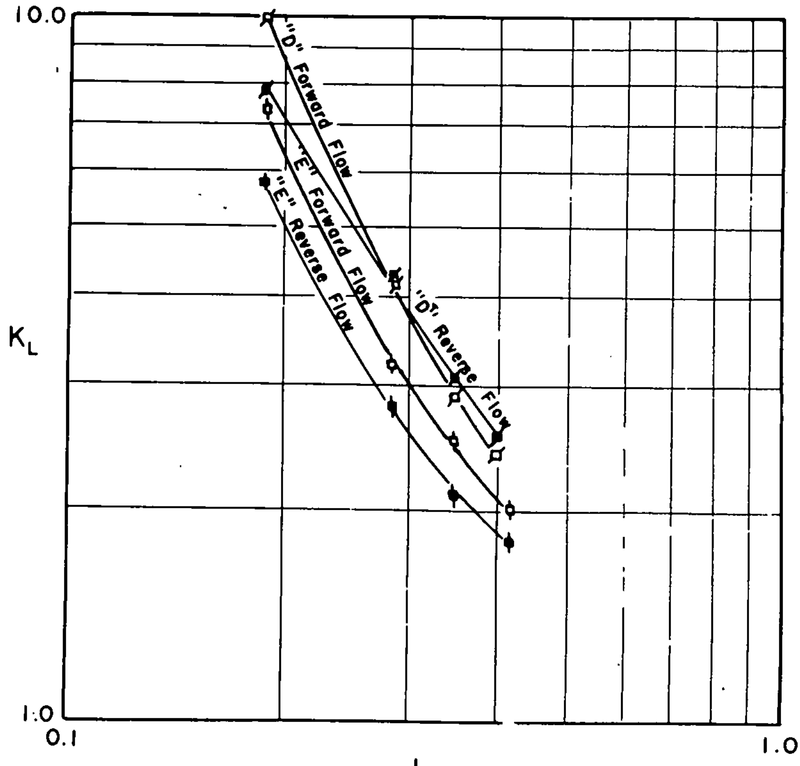


FIG. 14.—K_L vs. $\frac{L}{D}$, ANGLE STOP VALVES "D" AND "E".

in a gate valve because the bore length through the plug is relatively long, commonly about 1½ pipe diameters, insuring that at most angles of opening one expansion takes place within the bore. The K_L values are, from Fig. 12, approximately twice those for a circular orifice; this corresponds roughly to the double flow expansion.

Swing Check Valves

The test data for valves "A" and "M" are plotted in Fig. 13. Over most of the range of clapper angles, the K_L values are much lower than for the comparable orifice; this can be explained because

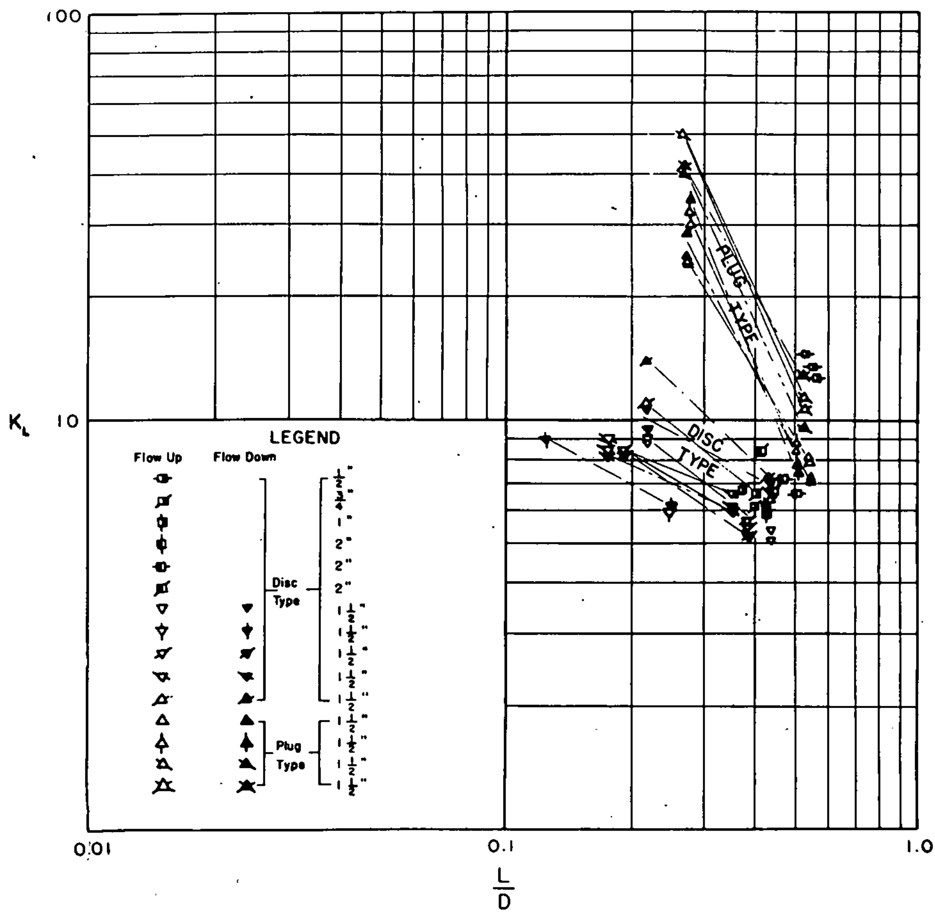


FIG. 15.—K_L VS. $\frac{L}{D}$, VARIOUS GLOBE VALVES.

the minimum through flow area of the stream as it passes the valve clapper is actually much larger than the projected visible, a. Also, as seen from Fig. 3, when the a/A ratio for these valves is unity there is still an expansion loss within the main valve body.

A common design value of K_L for swing check valves is 2.5. The curves of Fig. 13 show the wide variation in K_L which can be obtained as the clapper angle changes with flow rate. This plot does not show the effect of scale changes.

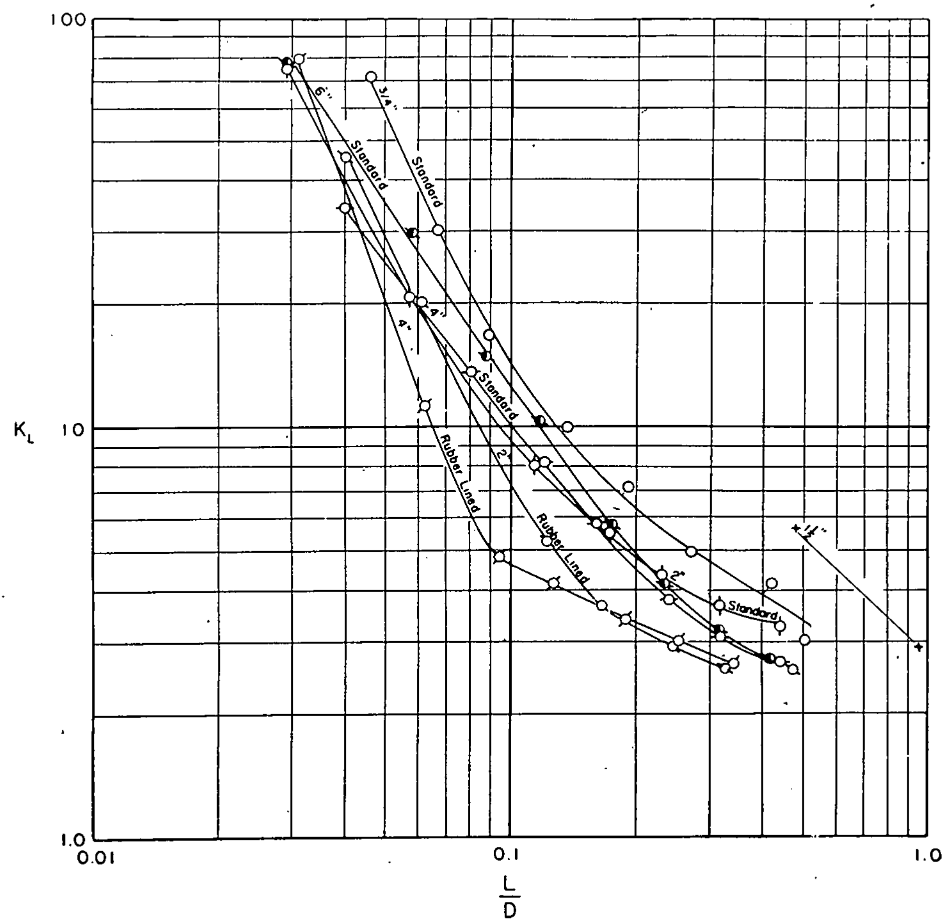


FIG. 16.—K_L VS. $\frac{L}{D}$, VARIOUS Y-PATTERN VALVES, FORWARD FLOW.

Ball Check Valves

With the 8-inch valve tested having a 10-inch diameter ball, there was no area a, as defined, for this valve. For valves of this general type, a ratio of minimum through flow area to pipe area could be computed, a being the annular gap around the ball with the latter in its forward flow position. There is no simple way of correlating different shapes of valve body.

Angle Stop Valves

Data for valves "D" and "E" are shown in Fig. 14. A common design figure for K_L for this type of valve, fully open, is 5.0. This is much higher (2-2½ times) than the results obtained from these valves. As stated, the K_L values for forward and reverse flows through the same valve at the same L/D ratio were surprisingly similar.

Globe Valves

Data for two types of globe valves, disc and plug, are shown in Fig. 15. The common value of K_L of 10 for a fully open globe valve is thus merely an average value which would apply more to the plug than to the disc type.

Y-Pattern Valves

Data for this type of valve are plotted in Fig. 16, for forward flows. As with the angle stop valves, K_L values are very comparable for the forward and reverse flows through the same valve at a given opening; the reverse flow values are a little larger. The K_L values shown here are also comparable to those found for angle stop valves "D" and "E."

CONCLUSIONS

The head loss characteristics of the different types of valves studied were not all placed on a common diagram. A comparison of the curves for which K_L is plotted against a/A shows that for valves of equal pipe size, the loss coefficient is higher for plug valves than for gate valves up to an a/A ratio of 0.65; for larger a/A ratios, as the valve position approaches fully open, the plug and ring follower gate valves show almost identical characteristics, with their K_L values becoming much less than that for the lifting disc-type gate valve. For larger sized valves, the butterfly type exhibits the lowest K_L values over the full range of valve openings.

It is hoped that the dimensionless plots involving the head loss coefficient for these various types of valves may prove useful in the design of hydraulic systems where it is desired to compute the head losses in components of the system as accurately as possible. The assemblage of data in one place should be a convenience in this regard.

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OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING
Boston Society of Civil Engineers

APRIL 20, 1955.—A Joint Meeting of the Boston Society of Civil Engineers with the Northeastern Section of the American Society of Civil Engineers was held this evening at the Boston City Club, 14 Court Square, Boston, Mass.

President Emory Ireland of the Northeastern Section, A.S.C.E. was presiding and called upon President Edwin B. Cobb of the B.S.C.E. to conduct any necessary business at this time.

President Cobb called upon the Secretary to announce election of new members and new applications for membership in the B.S.C.E.

President Ireland then introduced the speaker of the evening, General Frank H. Merrill, Commissioner of Public Works and Highways, State of New Hampshire, who spoke on "The National Highway Program".

Eighty-six members and guests attended the dinner preceding the meeting and 100 members and guests attended the meeting.

The meeting adjourned at 8:40 P.M.

ROBERT W. MOIR, *Secretary*

MAY 18, 1955.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the American Academy of Arts & Sciences, 28 Newbury Street, Boston, Mass., and was called to order by President Edwin B. Cobb, at 7:05 P.M.

President Cobb announced that the Minutes of the April 20, 1955 meeting would be published in a forthcoming issue of the JOURNAL and that the reading of the minutes would therefore be waived unless there was objection.

The President announced the death of the following members:—

Francis J. Mulvihill, who was elected a member March 18, 1924 and who died November 24, 1954.

George E. Dionne, who was elected a member January 21, 1952 and who died March 10, 1955.

Walter M. Fife, who was elected a member March 21, 1934 and who died April 22, 1954.

Frederic M. Rice, who was elected a member May 17, 1948 and who died May 7, 1955.

The Secretary announced the names of applicants for Membership in the B.S.C.E. and that the following had been elected to membership on May 16, 1955:—

Grade of Member—John C. Adams, Jr.,* Frank L. Parker,** Icchok Manusowicz, Leon A. Yacoubian

President Cobb then introduced the speakers of the evening, John W. Les-

*Transfer from Grade of Junior.
**Transfer from Grade of Student.

lie. Asst. Chief Engineering Div. New England Div. Corps of Engineers, and Anthony Minichiello, Asst. Chief, Military Branch, New England Div. Corps of Engineers who gave a most interesting illustrated talk on "The Portsmouth Air Force Base."

A short discussion period followed after which the President announced that a collation would be served in the Lounge on the floor above.

Seventy-five members and guests attended the meeting.

The meeting adjourned at 8:55 P.M.

ROBERT W. MOIR, *Secretary*

SANITARY SECTION ANNUAL OUTING

The Research Laboratory of the State Department of Public Health located at the Lawrence Experiment Station was inspected at 10:00 A.M. on Saturday, May 21, 1955, 17 persons attended.

Members and guests had lunch at the Andover Country Club. Luncheon was followed by short talks. Chairman Ariel A. Thomas introduced the speakers.

1. Joseph A. McCarthy, Chief of Laboratory, Lawrence Experiment Station, gave a brief outline of the activities and aims of the Experiment Station. During the tour of the Station in the morning, Mr. McCarthy had gone into detail in explaining the research and laboratory equipment and giving information on current studies.
2. Harold A. Thomas, Jr., Associate Professor of Sanitary Engineering, Harvard University, gave an outline of the experiments being conducted and the study being made of the capacity of streams to handle radio active wastes.

At 2:30 P.M. the group visited Cochickewick Ponds in North Andover where the field investigation is being conducted by Professor Thomas, and a demonstration was made of the field equipment being used.

The outing adjourned at 4:00 P.M.

JOHN F. FLAHERTY, *Clerk*

STRUCTURAL SECTION

APRIL 13, 1955.—A regular meeting of the Structural Section was held on this date at the Society Rooms. The meeting was called to order by the Chairman, Mr. C. J. Kray, at 7:05 P.M.

The speaker of the evening was Dr. A. A. Warlam, Consulting Engineer, Hastings, New York, formerly Associate Professor of Civil Engineering, New York University. His subject was "Recent Developments in Soil Testing Equipment".

Dr. Warlam described various field testing methods used by the military to determine the traffic capacity of unimproved soils. Such procedures are necessary to insure the mobility of military vehicles under combat conditions.

The devices used for this purpose include various types of manual penetrometers. These have the disadvantage that the operator may be exposed to enemy fire during the test. Dr. Warlam described a more recent development involving the use of airborne penetrometers which may be dropped from an airplane or projected by a rifle. Upon striking the surface this device measures the firmness of the ground and reports its finding by means of flares.

The talk was received with great interest by the 39 members in attendance.

JOHN M. BIGGS, *Clerk*

MAY 11, 1955. The regular monthly meeting of the Structural Section was held on this date at the Society Rooms and was called to order by Chairman, C. J. Kray, at 7:10 P.M.

The speaker was Mr. John C. Rundlett, Bridge Engineer, Department of Public Works, Commonwealth of Massachusetts. The subject was "Development of Prestressed Concrete in Bridge Construction."

Mr. Rundlett outlined the rapid development of prestressed concrete bridge construction in the United States. He discussed the various types and methods of construction which have been employed. The pioneering work done by the Massachusetts Department of Public Works in this field was described and illustrated with slides.

Mr. Rundlett also presented the results of tests on beams with various types of prestressing. It was concluded from these tests and other factors that bonding of the steel was advantageous. It was also concluded that prestressed concrete beams are very ductile but show excellent recovery after the load is removed.

The presentation was very well received by the 54 members in attendance.

JOHN M. BIGGS, *Clerk*

SURVEYING AND MAPPING SECTION

APRIL 7, 1954.—The twenty-third meeting of the Surveying and Mapping Section was held at the Society Rooms, at 7:15 P.M.

Prior to the meeting the Executive Committee of this Section met in the Society Rooms and discussed the program for the coming year.

The meeting was called to order by Chairman Wilbur C. Nylander. The minutes of the January 20th meeting were read by the Clerk and approved. There was no business to be taken up with the members.

The speaker of the evening was our Chairman Wilbur C. Nylander who was introduced by the Clerk. Mr. Nylander spoke on "History of the Middlesex Canal". He told of the difficulties in surveying a route from Lowell to Charlestown in 1793. The Canal was in operation for about 50 years, carrying freight and passengers to Lowell and up the Merrimack River to New Hampshire. He traced its route through from the Mill Pond in Charlestown to Somerville, Medford, Winchester, Woburn, Wilmington, Billerica to the Merrimack River in Lowell. Slides were shown of drawings and pictures obtained by Mr. Nylander from a Mr. Merrill of Wakefield. He also showed us a map and profile obtained from the Boston and Maine Railroad of the entire Canal.

After an interesting discussion by a number of the members, the meeting adjourned at 8:30 P.M.

There were 40 members and guests present.

LLEWELLYN T. SCHOFIELD, *Clerk*

MAY 19. 1954.—The twenty-fourth meeting of the Surveying and Mapping Section was held jointly with the Main Society at the Academy of Arts and Sciences, 28 Newbury Street, Boston, Mass.. at 7:00 P.M.

The meeting was called to order by President Miles N. Clair. Mr. Clair suggested waiving the reading of the minutes of the Main Society and called upon Chairman Wilbur C. Nylander of the Surveying and Mapping Section for any business to come before this meeting. Mr. Nylander stated that there was no business and suggested that the minutes of our previous meeting be waived. There being no objection, the minutes were waived.

President Clair introduced the speaker, Mr. Karl R. Kennison, Chief Engineer, Board of Water Supply of New York, who spoke on "problems that New York City Faces in Expanding its Water Supply Sources". Mr. Kennison illustrated his talk with slides and gave a very interesting report particularly as it was affected by the Supreme Court Decision affecting not only New York State, but New Jersey, Pennsylvania and Delaware.

After an interesting discussion by a number of the members and guests, the meeting was adjourned at 9:00 P.M., and a collation was enjoyed.

There were sixty-six members and guests present.

LLEWELLYN T. SCHOFIELD, *Clerk*

OCTOBER 20, 1954.—The twenty-fifth meeting of the Surveying and Mapping Section was held at the Society Rooms at 7:15 P.M.

The meeting was called to order by Chairman Wilbur C. Nylander. The minutes of the meetings held on April 7 and May 19, 1954 were read by the Clerk and approved. The Chairman, Mr. Nylander, reported receiving a letter from Charles Merrill Kelley, a member, relative to this section joining with the Massachusetts Association of Land Surveyors and Civil Engineers. This letter has been referred to the Board of Government by the Chairman. There was no business to be taken up with the members.

The speaker of the evening, John Clarkeson, President of Clarkeson Engineering Company, Inc. of Boston, was introduced by the Chairman. He spoke on "Use of Aerial Photogrammetry in the Location and Design of a Section of the Massachusetts Turnpike".

Mr. Clarkeson told how his company utilized photogrammetric surveys supplemented by ground surveys in the location and design features of a section of the Massachusetts Toll Road. He illustrated his remarks with actual plans as submitted by the photogrammetrist and plans, profiles and cross-sections prepared by his office of a section of the turnpike in Weston. Differences between photogrammetric and ground surveys were outlined in color on these plans. He told of the high order of accuracy of the field surveys for the control of the photography. He was satisfied that survey costs can be cut by more than 50 per cent by the use of photogrammetry over conventional survey methods as used on highway construction. The need for ground surveys at interchanges, bridge sites and wooded areas were stressed.

After an interesting discussion, the meeting was adjourned at 8:30 P.M.

There were fifty-two members and guests present.

LLEWELLYN T. SCHOFIELD, *Clerk*

JANUARY 26, 1955.—The twenty-sixth meeting of the Surveying and Mapping Section was held jointly with the Main Society at the Academy of Arts and Sciences at 7:00 P.M., on Wednesday, January 26, 1955.

The meeting was called to order by President Miles N. Clair. Upon completion of the business of the Society, Mr. Clair called upon Wilbur C. Nylander, Chairman of the Surveying and Mapping Section, to conduct the business of this section. Mr. Nylander waived the reading of the minutes of the previous meeting. The Nominating

Committee, consisting of Louis A. Chase, Charles M. Anderson and John J. Vertic, Chairman, submitted their report which was accepted. Nominations were closed and upon a motion duly made and seconded, it was voted that the Clerk cast one ballot for the slate of officers recommended. The Clerk cast one ballot for the following who were thereupon declared elected.

Chairman—Alexander J. Bone

Vice Chairman—
Llewellyn T. Schofield

Clerk—Ernest A. Herzog

Executive Committee—Nelson W. Gay, George A. McKenna, Roy L. Wooldridge

The speakers of the evening, Mr. William F. Uhl, Mr. George R. Rich and Mr. Hare substituting for Mr. Hall of the firm of Uhl, Hall and Rich, an affiliate of the Charles T. Main, Inc., were introduced by President Miles N. Clair. They delivered papers on the hydro-electric development of the St. Lawrence River.

Mr. Uhl outlined the scope of the joint project of the New York Power Authority and the Ontario Hydro-Electric Power Commission in harnessing the overflow of the Great Lakes at the head waters of the St. Lawrence River. He told how this project would cost $561,730,000.

Mr. Rich illustrated his paper with slides showing the proposed dams and their relationship to each other.

Mr. Hall's paper told of the preparation of survey data for the site and the co-ordination affected with our Canadian neighbors.

Upon completion of these papers an interesting discussion was held and the meeting adjourned at 8:25 P.M. A collation was enjoyed after the meeting.

There were one hundred ninety-four members and guests present.

LLEWELLYN T. SCHOFIELD, *Clerk*

ADDITIONS

Members

Floyd E. Brown, 119 Howard Street, Reading, Mass.

George F. Cuddy, 167 Central Street, Holliston, Mass.

Carol R. Gay, 329 Gallivan Blvd., Dorchester, Mass.

George C. Gay, 329 Gallivan Blvd., Dorchester, Mass.

William T. Gay, Jr., 490 High Street, Westwood, Mass.

Olcott L. Hooper, 228 Highland Avenue, Winchester, Mass.

Robert T. Koopman, 1515 V.F.W. Pky., W. Roxbury, Mass.

Charles R. Kurz, 4 Kirby Street, So. Dartmouth, Mass.

William B. S. Leong, 115 Gainsboro Street, Boston, Mass.

Logan T. Murdock, Moore Road, Wayland, Mass.

Peter S. Nelson, 3 Beverly Hills Ave., Beverly, Mass.

Victor G. Otto, 69 High Street, Newburyport, Mass.

David H. Owen, 44 First Street, Taunton, Mass.

Paul T. Quinton, 12 Lafayette Street, Quincy, Mass.

Euplio E. Rossi, 36 Hale Street, Boston, Mass.

Milton Sherman, 413 Conant Road, Weston, Mass.

DEATHS

George E. Doanne, Mar. 10, 1955
Walter M. Fife, Apr. 22, 1955
Francis J. Mulvihill, Nov. 24, 1954
Frederic M. Rice, May 7, 1955

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